California High-Speed Rail Authority

RFP No.: HSR 13-57

Request for Proposal for Design-Build Services for Construction Package 2-3

Book IV, Part G.2 – Geotechnical Baseline Report for Bid

CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

Preliminary Engineering for Procurement Record Set Submission

Fresno to Bakersfield

Sierra Subdivision Construction Package 2-3 Geotechnical Baseline Report for Bid

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

CALIFORNIA
High-Speed Rail Authority

Preliminary Engineering for Procurement
Record Set Submission
Fresno to Bakersfield
Sierra Subdivision
Construction Package 2-3
Geotechnical Baseline Report for Bid

Prepared by:

URS/HMM/Arup Joint Venture

April 2014

Preliminary Engineering for Procurement
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April 2014

This report has been prepared under the direction of the following Geotechnical Engineer.

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RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Table of Contents

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Tables

Figures

Appendices

List of Abbreviations and Acronyms

AASHTO	American Association of State Highway Transportation Officials
Authority	California High-Speed Rail Authority
API	American Petroleum Institute
ASTM	ASTM International (formerly American Society for Testing and Materials)
Authority	California High-Speed Rail Authority
bgs	below ground surface
Caltrans	California Department of Transportation
CHSTP	California High-Speed Train Project
CIDH	cast-in-drilled-hole
cm	centimeter
CP2-3	Construction Package 2-3
CPT	cone penetration test
E_s	Soil Modulus
FB	Fresno to Bakersfield
ft	feet
g	gravitational acceleration, 9.81 meters / second^2
GAP	Ground Assumptions for Procurement
GBR-B	Geotechnical Baseline Report for Bid
GBR-C	Geotechnical Baseline Report for Construction
GDR	Geotechnical Data Report
GI	ground investigation
GSHR	Geologic and Seismic Hazards Report
HMM	Hatch Mott MacDonald
HSR	high-speed rail
JPL	Jet Propulsion Laboratories
k_h	modulus of horizontal subgrade reaction
k'_v	modulus of vertical subgrade reaction
kN	kilonewton
MCE	maximum considered earthquake
mi	miles
mm	millimeters
M_W	moment magnitude
$(N_1)_{60}$	standard penetration test N-value corrected for overburden and field procedures
N_{60}	standard penetration test N-value corrected for hammer energy
NA	not available
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resources Conservation Service
OBE	operating basis earthquake
OSHA	Occupational Safety and Health Administration
ppm	parts per million
PE4P	preliminary engineering for procurement
q_c	CPT tip resistance
q_t	CPT tip resistance corrected for pore water effects
SBT_N	normalized CPT soil behavior type
sec	second
SJV	San Joaquin Valley
SPT	standard penetration test
SR	State Route
T	period
TM	Technical Memorandum
umhos	micromhos

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USGS	United States Geological Survey
V_s, V_{s30}	shear wave velocity, average shear wave velocity in the upper 30 meters
WEAP	wave equation analysis of piles

Section 1.0

Introduction

1.0 Introduction

In 1996, the state of California established the California High-Speed Rail Authority (Authority). The Authority is responsible for studying alternatives to construct a rail system that will provide intercity high-speed rail (HSR) service on over 800 miles of track throughout California. This rail system will connect the major population centers of Sacramento, the San Francisco Bay Area, the Central Valley, Los Angeles, the Inland Empire, Orange County, and San Diego. The Authority is coordinating the project with the Federal Railroad Administration. The California High-Speed Train Project (CHSTP) is envisioned as a state-of-the-art, electrically powered, high-speed, steel-wheel-on-steel-rail technology that will include state-of-the-art safety, signaling, and automated train-control systems.

The statewide CHSTP has been divided into sections for the planning, environmental review, coordination, and implementation of the project. This Geotechnical Baseline Report for Bid (GBR-B) is focused on the section of the CHSTP between Fresno and Bakersfield, specifically the Construction Package 2-3 (CP2-3), which extends from E American Avenue south of the Fresno metropolitan area to approximately 1 mile north of the border between Tulare County and Kern County. During the initial planning process, the CHSTP alignment alternatives are dynamic and subject to revision.

1.1 Geotechnical Contract Documents

The key geotechnical documentation provided in the Contract Documents is this Fresno to Bakersfield (FB) CP2-3 GBR-B. The FB CP2-3 Geotechnical Data Report (GDR) (URS/HMM/Arup 2014), erratum to the GDR, and the Geologic and Seismic Hazards Report (GSHR) are also available as reference documents. The CP2-3 GDR provides details of the ground investigation (GI) such as drilling procedures, soil sampling, in situ testing, hydrogeologic testing, and historical geotechnical information gathered prior to the exploration phase. The CP2-3 GDR also includes exploration logs, details pertaining to laboratory testing, procedures used to conduct various index tests, strength and deformation tests, test results, and a limited environmental assessment. Definitions for terms used in both the CP2-3 GBR-B and CP2-3 GDR are contained in Section 11.0 Glossary.

This CP2-3 GBR-B and the referenced CP2-3 GDR cover only the FB CP2-3 corridor.

1.1 Purpose

The principal purpose of this CP2-3 GBR-B is to set baselines for ground conditions to facilitate the bidding process such that all bidders can rely on a single contractual interpretation of the geotechnical conditions when preparing their bids. This report summarizes anticipated ground conditions for construction of the CP2-3 alignment, which extends between E American Avenue and about 1 mile north of the Tulare/Kern county line.

This GBR-B is a representation of the conditions upon which the design-build Contractor may rely for bidding. GIs conducted in preparation of the CP2-3 GDR are considered preliminary and shall not be solely relied on for final design. It is incumbent upon the Contractor to conduct supplemental investigations adequate to complete final design and prepare a Geotechnical Baseline Report for Construction (GBR-C). The CP2-3 GBR-C will serve as the basis of resolution for differing site conditions during construction. The CP2-3 GBR-B has been prepared such that it will be superseded by the CP2-3 GBR-C, and the CP2-3 GBR-C will incorporate additional geotechnical exploration data and analyses. The CP2-3 GBR-C will become the basis of final design and construction conditions.

The engineering judgment applied in the interpolations and extrapolations of information contained in the CP2-3 GDR reflect the view of the Authority in establishing the baseline conditions. The baseline conditions presented in this report will (1) serve as a baseline for geotechnical conditions anticipated to be encountered and (2) assist the Contractor in evaluating the requirements for installation of foundation elements and excavating and supporting the ground.

1.2 Report Structure

This report has been prepared in general accordance with Technical Memorandum (TM) 2.9.2 Geotechnical Reports Preparations Guidelines and the latest edition of the American Society of Civil Engineers' publication *Geotechnical Baseline Reports for Construction: Suggested Guidelines* (Essex 2007). Sections 1.0 through 5.0 provide background information, while Sections 6.0 through 9.0 provide specific recommendations related to ground characterization and behavior. Sections 10.0 and 11.0 provide reference information.

Section 1.0 provides an introduction to the project including project location, report purpose, and organization. Section 2.0 provides a project description including key project features and existing man-made structures of significance to the project. Section 3.0 describes sources of geotechnical information including prior geotechnical reports, TMs, data from desk studies, and data from the PE4P GI for CP2-3. Section 4.0 describes the project setting through physiography, geology, seismicity, and hydrogeology; Section 5.0 describes previous construction experience in the project vicinity.

Section 6.0 presents ground characterization and geotechnical baselines, Section 7.0 describes design considerations for the various proposed structures, Section 8.0 describes construction considerations, and Section 9.0 describes recommended instrumentation and monitoring during construction.

Section 10.0 is a list of documents referenced in this report; Section 11.0 is a glossary of terms used in this report.

1.3 Basis of Report

The baseline values in this report have been developed from geotechnical information and data gathered through desk studies and the PE4P CP2-3 GI, which included widely spaced exploratory boreholes, cone penetration tests (CPTs), and laboratory and field tests. The results from this investigation are presented in the CP2-3 GDR and erratum. Since access to exploration locations in Kings County was delayed and no current geotechnical information is available, the Authority requested that Kings County geotechnical baselines be presented under separate cover. Accordingly, the Contractor is directed to review the Kings County Ground Assumptions for Procurement (GAP) report for the Kings County section of the alignment.

1.4 Project Constraints and Restrictions

The baseline recommendations in this report have been derived from the available data. Limited site access, limited historical data, and wide spacing of explorations constrain the recommendations to a level appropriate for preliminary engineering, not final design. PE4P structures were designed using geotechnical parameters from historical data only. However, when the CP2-3 GDR and this GBR-B became available, the assumptions made to complete the PE4P structures design using historical data were found to be reasonable when compared to the data collected and baselines developed herein.

No GI activities were conducted within Kings County limits at the time of this report since access has been delayed. Thus, the check of design assumptions in Kings County was not completed.

During construction, ground behavior will be influenced by the Contractor's selected design, equipment, means, methods, and level of workmanship. The Contractor must assess how these factors will influence ground behavior and baseline values provided in this report in consideration of the project as a whole.

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Section 2.0

Project Description

2.0 Project Description

2.1 Fresno to Bakersfield High-Speed Rail Section

The proposed FB Section of the HSR is approximately 114 miles long and traverses a variety of land uses, including farmland, large cities, and small cities. The FB Section includes viaducts, elevated structures, retaining walls and segments where the HSR will be at-grade or on embankment. The route of the FB Section passes by or through the rural communities of Bowles, Laton, Conejo, Armona, and Allensworth and the cities of Fresno, Hanford, Corcoran, Wasco, Shafter, and Bakersfield.

The FB Section extends from north of Stanislaus Street in Fresno to the northernmost limit of the Bakersfield to Palmdale Section of the HSR at Oswell Street in Bakersfield.

2.2 Alignments

The FB Section is a critical link connecting the northern HSR sections of Merced to Fresno and the Bay Area to the southern HSR sections of Bakersfield to Palmdale and Palmdale to Los Angeles. The FB Section includes HSR stations in the cities of Fresno and Bakersfield, with a third station in the vicinity of Hanford. The Fresno and Bakersfield stations are this section’s project termini.

The FB Section of the HSR is divided into 10 subsections, most of which have multiple alternative alignments. Table 2.2-1 summarizes and Figure 2.2-1 illustrates the subsections and their corresponding alignments.

Table 2.2-1
FB Alignment Subsections

Alignment Prefix	Alignment Subsection Name	Location		County	EIR/EIS Name*
		Begin	End		
F1	Fresno	San Joaquin St	E Lincoln Ave	Fresno	BNSF
M	Monmouth	E Lincoln Ave	E Kamm Ave	Fresno	BNSF
H	Hanford	E Kamm Ave	Iona Ave	Fresno and Kings	BNSF (Hanford East)
HW	Hanford West Bypass	E Kamm Ave	Idaho Ave		Hanford West Bypass 1 & 2
HW2	Hanford West Bypass	E Kamm Ave	Iona Ave		Hanford West Bypass 1 & 2 Modified
K1	Kaweah	Idaho Ave	Nevada Ave	Kings	Hanford West Bypass 2 (at-grade) (connects to C1 [Corcoran Elevated] or C2 [Corcoran Bypass])
K2		Idaho Ave	Nevada Ave		Hanford West Bypass 1 (at-grade) (connects to C3 [BNSF through Corcoran])
K3		Iona Ave	Nevada Ave		BNSF (Hanford East) (connects to C3 [BNSF through Corcoran])
K4		Iona Ave	Nevada Ave		BNSF (Hanford East) (connects to C1 [Corcoran Elevated] or C2 [Corcoran Bypass])
K5		Iona Ave	Nevada Ave		Hanford West Bypass 2 Modified (below-grade) (connects to C1 [Corcoran Elevated] or C2 [Corcoran Bypass])
K6		Iona Ave	Nevada Ave		Hanford West Bypass 1 Modified (below-grade) (connects to C3 [BNSF through Corcoran])
C1	Corcoran	Nevada Ave	Ave 128	Kings and Tulare	Corcoran Elevated
C2	Corcoran Bypass	Nevada Ave	Ave 128		Corcoran Bypass
C3	Corcoran	Nevada Ave	Ave 128		BNSF (through Corcoran)
P	Pixley	Ave 128	Ave 84	Tulare	BNSF
A1	Allensworth Bypass	Ave 84	Elmo Hwy	Tulare and	Allensworth Bypass

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Table 2.2-1
FB Alignment Subsections

<table>
<tr><th rowspan="2">Alignment Prefix</th><th rowspan="2">Alignment Subsection Name</th><th colspan="2">Location</th><th rowspan="2">County</th><th rowspan="2">EIR/EIS Name*</th></tr>
<tr><th>Begin</th><th>End</th></tr>
<tr><td>A2</td><td>Through Allensworth</td><td>Ave 84</td><td>Elmo Hwy</td><td>Kern</td><td>BNSF (through Allensworth)</td></tr>
<tr><td>L1</td><td rowspan="4">Poso Creek</td><td>Elmo Hwy</td><td>Whisler Rd</td><td rowspan="4">Kern</td><td>Allensworth Bypass (connects to BNSF [through Wasco-Shafter])</td></tr>
<tr><td>L2</td><td>Elmo Hwy</td><td>Poplar Ave</td><td>Allensworth Bypass (connects to Wasco-Shafter Bypass)</td></tr>
<tr><td>L3</td><td>Elmo Hwy</td><td>Whisler Rd</td><td>BNSF (through Allensworth) (connects to BNSF [through Wasco-Shafter])</td></tr>
<tr><td>L4</td><td>Elmo Hwy</td><td>Poplar Ave</td><td>BNSF (through Allensworth) (connects to Wasco-Shafter Bypass)</td></tr>
<tr><td>WS1</td><td>Through Wasco-Shafter</td><td>Whisler Rd</td><td>Hageman Rd</td><td rowspan="2">Kern</td><td>BNSF (through Wasco-Shafter)</td></tr>
<tr><td>WS2</td><td>Wasco-Shafter Bypass</td><td>Poplar Ave</td><td>Hageman Rd</td><td>Wasco-Shafter Bypass</td></tr>
<tr><td>B1</td><td>Bakersfield Urban</td><td>Hageman Rd</td><td>Baker St</td><td rowspan="3">Kern</td><td>BNSF (Bakersfield North)</td></tr>
<tr><td>B2</td><td>Bakersfield Urban</td><td>Hageman Rd</td><td>Baker St</td><td>Bakersfield South</td></tr>
<tr><td>B3</td><td>Bakersfield Urban</td><td>Hageman Rd</td><td>Baker St</td><td>Bakersfield Hybrid</td></tr>
<tr><td colspan="6">*Environmental Impact Report/Statement</td></tr>
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RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

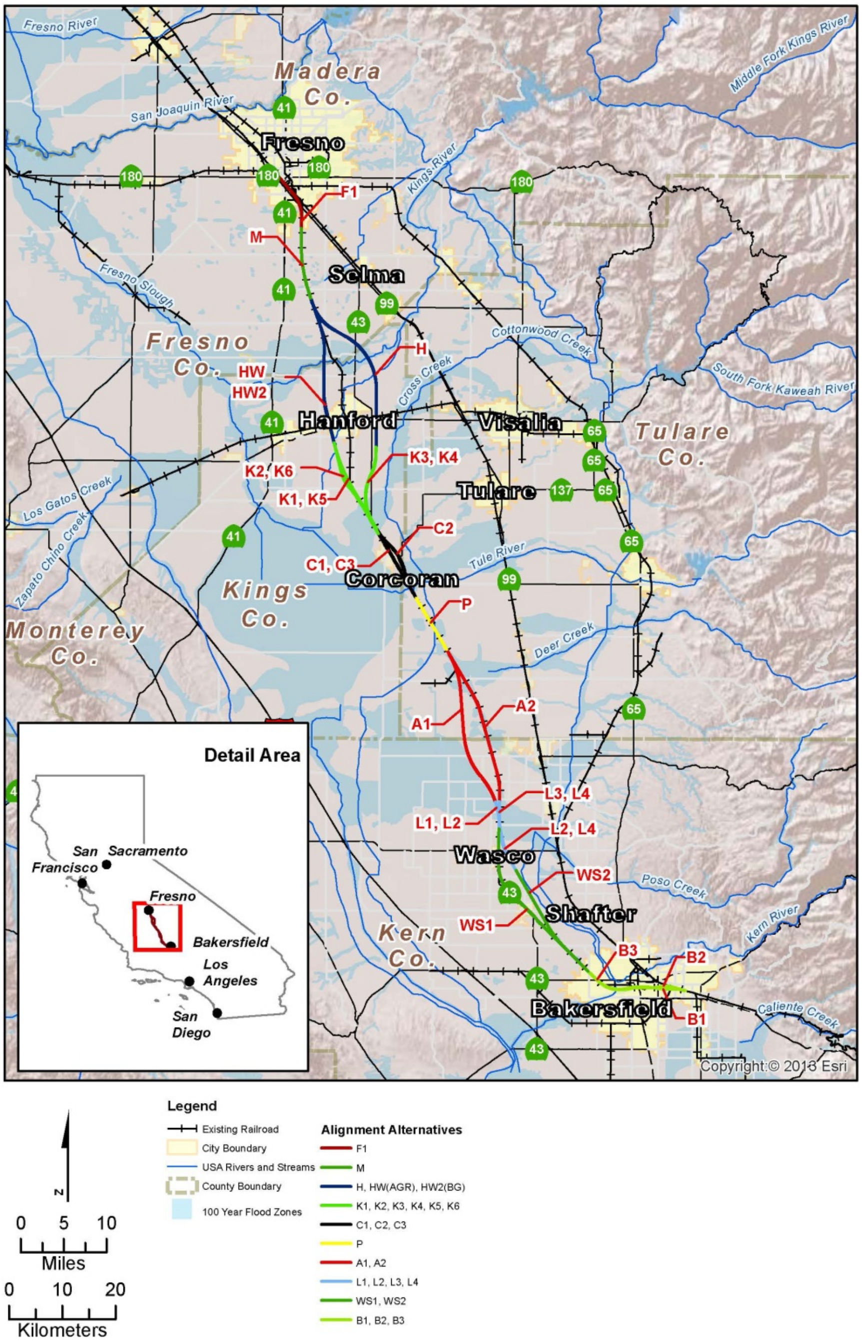

Figure 2.2-1
Overview of Alignments

2.2.1 CP2-3 Alignment Features

The CP2-3 alignment spans approximately 64 miles traversing Fresno, Kings, and Tulare Counties along the F1, M, H, K4, C2, P, and A1 Subsections. Figure 2.2-2 shows the CP2-3 alignment, with the preferred alignment labeled. The alignment begins just south of E American Avenue and follows alongside the BNSF railroad south of Fresno to Conejo, then continues southeast across agricultural land, eventually crossing over State Route 43 (SR 43) about 1.3 miles south of Davis Avenue. Crossing the Kings River Complex on elevated structure from SR 43 to the south side of Kings River, the alignment then crosses through the eastern side of Kings County and Hanford, paralleling the east side of 7-1/2 Avenue then curves to the southwest, crossing SR 43 about 1/3 of a mile south of Jersey Avenue. Rejoining the east side of the BNSF railroad north of Corcoran, the alignment then bypasses Corcoran to the east and enters Tulare County southeast of Corcoran. In Tulare County, the alignment follows along the west side of SR 43 until it curves to the southwest near Deer Creek. The alignment then passes about 1.25 miles west of Allensworth, terminating about 1 mile north of the Kern County line.

The CP2-3 alignment includes at-grade and embankment rail sections as well as bridges and viaducts. This contract also includes numerous secondary transverse vehicular and pedestrian bridges at select local street intersections. The design requires shallow and deep foundations, retaining walls, and earthwork embankments for the proposed improvements. The key project features are described in Table 2.2-2, from north to south. The table has been populated with the current 15% design structures from the FB CP2-3 PE4P Structures Report (URS/Arup/HMM 2014), which is subject to modification after compilation of this report. Consult the Contract Documents for the most updated information.

Table 2.2-2
Significant Structures – CP2-3

Name	Approximate Start Station (ft)	Approximate End Station (ft)	Description of Location	Approximate Length (ft)
At-grade	577+00	1086+00	From E American Ave to south of Willow Ave	50,900
Retained	1086+00	1105+70	From south of Willow Ave to north of Conejo Ave	1,970
Aerial	1105+70	1156+20	From north of Conejo Ave to south of Peach Ave	5,050
Retained	1156+20	1173+50	From south of Peach Ave to north of Clarkson Ave	1,730
At-grade	1173+50	1439+19	From north of Clarkson Ave to north of Highland Ave	26,570
Retained	1439+19	1463+58	From north of SR 43 to south of SR 43	2,440
Elevated[F/K]	1463+58	1596+56	From SR 43 to south of Kings River	13,300
Retained	1596+56	1622+50	From south of Kings River to north of Douglas Ave	2,590
At-grade	1622+50	1885+40	From north of Douglas Ave to north of Fargo Ave	26,290

Table 2.2-2
Significant Structures – CP2-3

Name	Approximate Start Station (ft)	Approximate End Station (ft)	Description of Location	Approximate Length (ft)
Retained	1885+40	1903+57	From north of Fargo Ave to north of Grangeville Blvd	1,820
Aerial	1903+57	2008+37	From north of Grangeville Blvd to south of SR 198	10,480
Retained	2008+37	2023+48	From south of SR 198 to north of Hanford Armona Rd	1,510
At-grade	2023+48	2240+32	From north of Hanford Armona Rd to SR 43	21,680
Bridge	2240+32	2246+06	From SR 43 to SR 43	570
At-grade	2246+06	2436+00	From SR 43 to south of Tulare Ave	18,990
Retained	2436+00	2446+81	From south of Tulare Ave to south of Tulare Ave	1,080
Aerial	2446+81	2538+71	From south of Tulare Ave to SR 43	9,190
Retained	2538+71	2583+63	From SR 43 to SR 43	4,490
At-grade[K/T]	2583+63	2966+50	From SR 43 to south of Ave 152	38,290
Retained	2966+50	2989+36	From south of Ave 152 to north of Ave 144	2,290
Aerial	2989+36	3046+02	From north of Ave 144 to Tule River	5,670
Retained	3046+02	3064+70	From Tule River to south of Ave 136	1,870
At-grade	3064+70	3982+20	From south of Ave 136 to north of Deer Creek	91,750
Retained	3982+20	4005+25	From north of Deer Creek to Deer Creek	2,310
Aerial	4005+25	4067+65	From Deer Creek to south of Stoil Spur	6,240
Retained	4067+65	4085+95	From south of Stoil Spur to south of Stoil Spur	1,830
At-grade[T]	4085+95	4435+50	From south of Stoil Spur to north of Kern County Line	34,960

[F/K] Transition from Fresno County to Kings County occurs at 1508+80 (H) along the preferred alignment.
[K/T] Transition from Kings County to Tulare County occurs at 2856+00 (C2) along the preferred alignment.
[T] Alignment terminates in Tulare County, 1 mile north of the border with Kern County, 4488+35 (A1).

The CP2-3 GI discussed in the CP2-3 GDR focused on the preferred alignment consisting of F1, M, H, K4, C2, P, and A1 alignments within the limits of CP2-3, shown in color in Figure 2.2-2.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

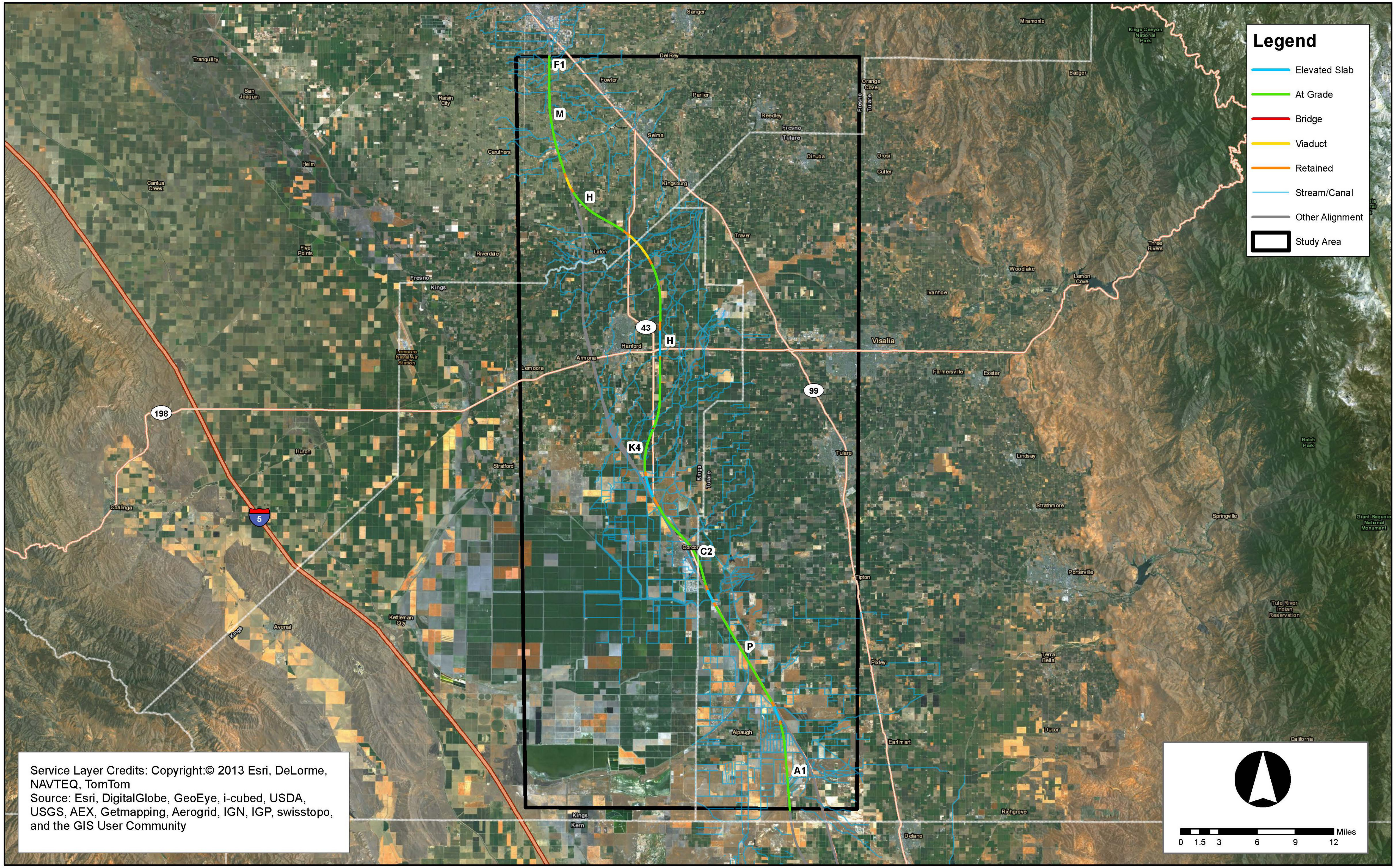

Figure 2.2-2
Vicinity Map of CP2-3 Alignment

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Section 3.0

Sources of Geologic and Geotechnical Information

3.0 Sources of Geologic and Geotechnical Information

3.1 Project Sources

Data and information for this report were primarily obtained from publically available reports and results of the PE4P GI. The sources include the following:

- FB Archeological Survey (URS/HMM/Arup 2011).
- FB Geology, Soils, and Seismicity Technical Report (URS/HMM/Arup 2012).
- FB PE4P Record Set CP2-3 GDR (URS/HMM/Arup 2013a).
- FB 15% Record Set GI Work Plan (2013b).
- FB Draft Environmental Impact Statement/Report (URS/HMM/Arup 2013c).
- FB 15% Record Set Geologic and Seismic Hazards Report (GSHR; URS/HMM/Arup 2013d).
- FB PE4P record Set Hydrology, Hydraulics, and Drainage Report (URS/HMM/Arup 2013e).
- FB 15% Record Set Utility Impact Report (URS/HMM/Arup 2013f).

3.2 Site Investigations

The PE4P GI for CP2-3 was conducted between August 19 and November 13, 2013, and consisted of drilling 19 rotary-wash boreholes and performing 74 CPTs. Soil samples were collected from boreholes at 5-foot intervals using standard penetration test (SPT) samplers and California Modified samplers driven with automatic hammers. Energy calibration tests were performed on the automatic hammers used during the exploration program.

In situ testing performed during the investigation included shear wave velocity (V_s) profiles in three boreholes using the suspension velocity logging method, V_s profiles in six CPTs, and pore water pressure dissipation tests in all 74 CPTs. Five boreholes, S0020R, S0029R, S0068R, S0071R, and S0072R, were converted to standpipe piezometers to monitor groundwater-level fluctuations In situ testing performed during the exploration program also included SPTs and pocket penetrometer and torvane testing on retrieved samples.

Laboratory testing was performed on representative soil samples to obtain index and engineering properties. Geotechnical index testing included moisture content, density, No. 200 sieve wash, hydrometer, grain-size analysis, specific gravity, Atterberg limit, and organic content tests. Laboratory testing for engineering properties included direct shear, triaxial undrained and drained, consolidation, compaction, California bearing ratio, and corrosion test methods. Soil corrosivity was tested for by resistivity, pH, sulfate content, and chloride content methods.

3.3 Historical Investigations

The primary source of publicly available historical geotechnical data collected during 15% design was from the California Department of Transportation (Caltrans) database of as-built construction records.

Caltrans data are concentrated along SR 41, 43, and 99, from projects dating between 1953 and 1997. For each project, several boreholes were drilled, logged, and plotted on a cross section. None of the Caltrans records contain laboratory test data. Borehole records collected from Caltrans extend to a maximum depth of 122 feet below ground surface (bgs), with an average borehole depth of 42 feet bgs. Historical Caltrans data are included in Appendix A of the GDR.

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Section 4.0

Physiography and Geology Overview

4.0 Physiography and Geology Overview

The section provides a brief description of physiography, geology, and seismicity within the CP2-3 corridor. Detailed discussion of physiography, geology, and seismicity along the entire FB alignment is presented in the GSHR.

4.1 Physiography

The CP2-3 alignment is located within the southern portion of the 450-mile-long Great Valley Geomorphic Valley (Bartow 1991). The topography of the Great Valley (the southern portion of which is referred to as the San Joaquin Valley [SJV]) is relatively flat. The SJV is bordered by the Pacific Coast Range to the west, the Klamath Mountains and Cascade Range to the north, the Sierra Nevada to the east, and San Emigdio and Tehachapi mountains to the south.

Superimposed upon this large-scale, relatively flat topography is a localized topography caused by recent incisions of river systems. This localized topography is composed of short, steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees) or narrow gully-type valleys, depending on their age and the amount of flow; however, along the CP2-3 alignment these features appear to have been either channelized or redirected along more convenient routes to accommodate the present urbanization.

The topography along the CP2-3 corridor is generally flat and varies between elevation 295 and 205 feet relative to the North American Vertical Datum of 1988. Localized variations on the ground surface elevation occur at existing road embankments, river and other hydraulic crossings, detention basins, and other man-made features such as irrigation canals and road and rail crossings.

4.2 Geologic Setting

4.2.1 Regional Geology

In his discussion of geology in the southern SJV, Bartow (1991) writes that the SJV is an "asymmetric structural trough that is filled with prism sediments up to 30,000 feet thick. It formed the southern part of an extensive fore-arc basin that evolved during the Cenozoic into today's hybrid intermontane basin."

Bartow (1991) continues discussing the sedimentation infill of the SJV, stating that it

> evolved through the gradual restriction of the marine basin due to uplift and emergence of the northern Great Valley in the late Paleogene, the closing off of the western outlets in the Neogene, and finally the sedimentary infilling in the Neogene and Quaternary. These sediments rest on crystalline basement rocks of the southwestward-tilted Sierran block.

4.2.2 Local Geology

Subsurface soils have been characterized into three separate layers: (1) Existing Fill, (2) Alluvial Fan, and (3) Lacustrine Deposits. Based on the geologic maps (Jenkins 1964 1965; Page 1986), Alluvial Fan units are prevalent in Fresno County while Lacustrine Deposits are more prevalent in Tulare County.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

The Alluvial Fan is more prevalent throughout Fresno County and includes the Modesto (Qf) and Riverbank (Qc) Formations as well as the Basin (Qb), Stream Channel (Qsc), and Sand Dunes (Qsd) deposits. A distinction was not made between these units because the GI did not identify a discernible difference between their composition and engineering properties.

The Lacustrine (Ql) Deposits are more prevalent in Tulare County, where they have been mapped within the boundaries of the former Tulare Lake Bed. In general these units are softer and comprise sandy silts and sandy clay soils, with some deposits of fat clay.

This report avoids the use of formation names to identify stratigraphic or mechanical baseline properties because of the lack of agreement across many geological references on formation depths, extents, nomenclature, and identifying characteristics.

4.3 Seismic Setting

According to Jennings (1994), the Fresno area is located "within a relatively seismically quiescent region between two areas of documented tectonic activity, the Coast Ranges-Sierran Block boundary zone to the east and the Pacific Coast Ranges boundary zone to the west."

Jennings (1994) identifies the predominant source of seismic shaking in the SJV as the Pacific Coast Ranges, which contain "many active faults that are associated with the northwest-trending San Andreas Fault System." The San Andreas Fault System is the principal tectonic element of the North American-Pacific plate boundary in California.

4.3.1 Faults and Seismicity

There are no known active faults crossing or within close proximity to the alignment within the study area. The San Andreas Fault, located approximately 45 miles west of the CP2-3 alignment from the site, has the highest slip rate and is the most seismically active of any fault near the HSR alignment. The closest fault to the alignment is the Clovis Fault; the potential seismicity of this fault has not been characterized in the literature reviewed. While they do not cross the CP2-3 alignment, the San Andreas, White Wolf, Garlock, Kern Canyon, Edison, and Tehachapi Creek Faults are deemed "capable" by HSR standards (FB GSHR 2013d).

There are a number of other faults capable of producing large-magnitude earthquakes near the HSR alignment. A list of known faults within 100 miles of the study area and their characteristics is presented in Table 4.3-1. These faults are shown in Figure 4.3-1 along with other mapped Quaternary faults in the vicinity of the study area. The Corcoran Clay Fault Zone is not highlighted on Figure 4.3-1 as its activity is associated with subsidence caused by an erosion of the mantle beneath Tulare Lake known as the Tulare Lake Tectonic Drip Zone. This zone does not appear to be seismically active nor do these faults appear on any USGS or CGS published maps.

Table 4.3-1
Characteristics of Faults within 100 miles of the Study Area (USGS 2006)

Fault Name	Fault Type	Slip Rate (mm/yr)	Distance and Bearing to FB HSR Alignment
San Andreas	Right-Lateral Strike-Slip	20–35	47 miles (or more) W of alignment
Great Valley (Segments 10–14)	Blind Thrust	1.5	25–35 miles (or more) W of alignment
Ortigalita	Right-Lateral Strike-Slip	0.5 to 1.5	64 miles W of Fresno
San Joaquin	Reverse	–	57 miles W of Fresno, slightly E of Ortigalita Fault
O'Neill	Reverse	–	58 miles W of Fresno, slightly E of Ortigalita Fault
Nunez	–	–	48 miles W of Corcoran
Foothills	Normal	0.1	90 miles NW of Fresno; 40 miles E of Stockton
Round Valley/Hilton Creek	Normal	1	80 miles NE of Fresno
Clovis Fault	–	–	12 miles E of Clovis
Corcoran Clay Fault Zone (Not shown on Fig 4.3-1)	Normal	–	spanning across the HSR alignment from Hanford to the Kern/Tulare County line
Owens Valley	Right-Lateral Strike-Slip	1.5	85 miles E of alignment
Kern Canyon	Normal	–	66 miles E of alignment at Hanford
Kern Front	Normal	–	30 miles SE of Tule River Crossing
Kern Gorge	Normal	–	14 miles NE of Bakersfield
Buena Vista	Thrust	–	50 miles S of alignment
Southern Sierra Nevada (Independence Section)	Normal	0.1	80 miles W of alignment
Oil Field Fault Zone (North)*	Normal	–	2.25 miles N of alignment
Oil Field Fault Zone (South)*	Normal	–	0.75 miles N of alignment
Garlock	Left-Lateral Strike-Slip	2–10	34 miles SE of alignment
White Wolf	Left-Lateral Reverse	3–8.5	13 miles SE of alignment
Breckenridge	Normal	–	18 miles E of alignment
Poso Creek/Pond	Normal	–	0/2 miles E of alignment
Wheeler/Pleito	Normal	1.4	30 miles S of alignment
Edison Fault	Normal	–	Crosses the HSR alignment east of Bakersfield
Southern Sierra Nevada (Haiwee Reservoir)	Normal	7–14	44 miles E of alignment

– No information available

*These faults appear on the Caltrans 1996 Seismic Hazards Map but have apparently have been de-rated since they do not appear on the Caltrans 2007 Deterministic Peak Ground Acceleration Map.

Source: SCEC 1999, WGCEP 2007, Caltrans 2007, USGS, CGS 2010

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

CALIFORNIA High-Speed Rail Authority

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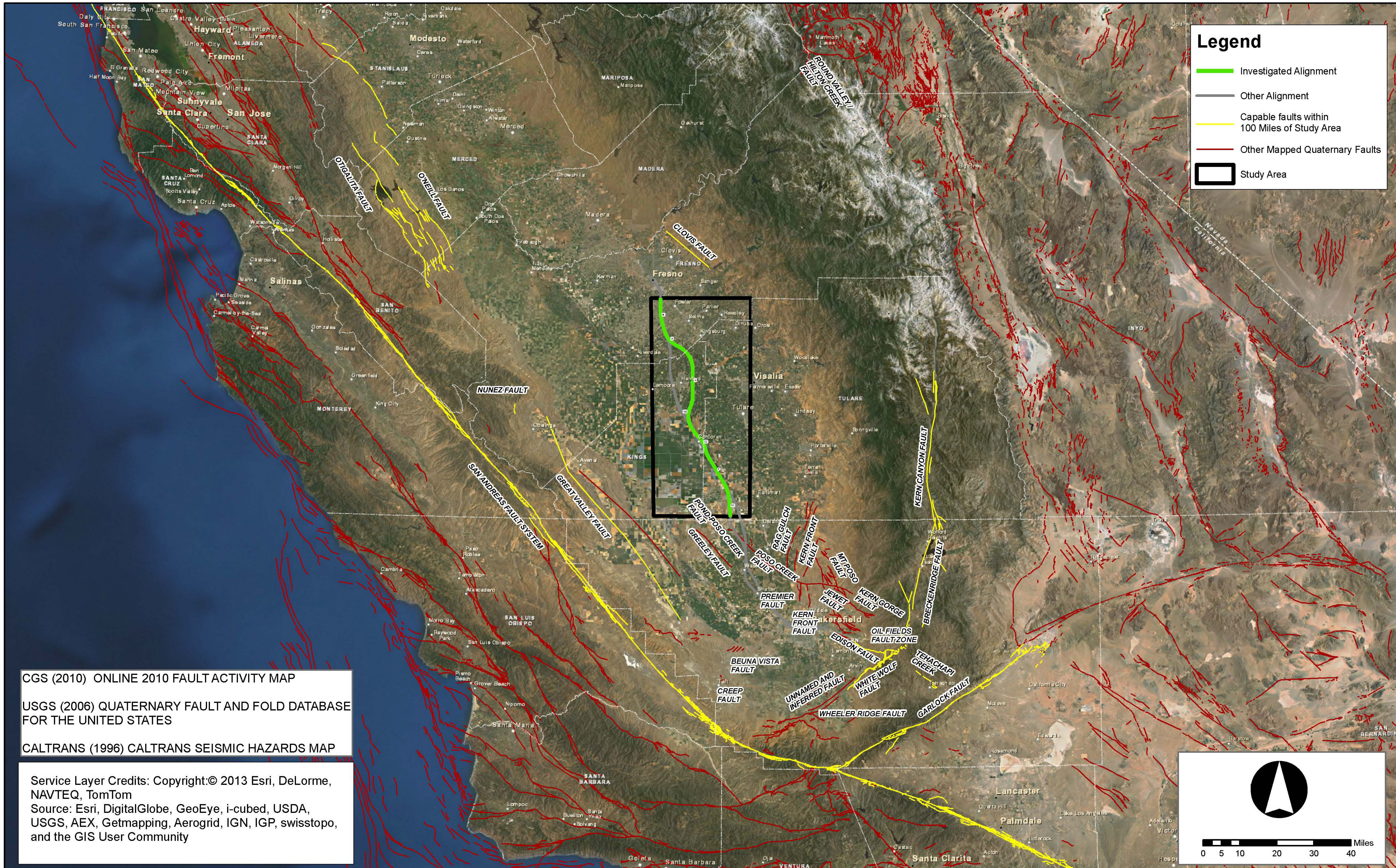

Figure 4.3-1
Mapped Faults in Vicinity of Study Area

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

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4.3.2 Design Earthquake and Design Ground Motion

For the CP2-3 alignment, two design-level earthquakes have been defined for final design per the Design Criteria Manual:

> **Maximum considered earthquake (MCE)** – ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to maximum moment magnitude [M_w]) of any fault in the vicinity of the structure.
>
> **Operating basis earthquake (OBE)** – ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

Site-specific spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced and Bakersfield were developed for preliminary engineering. Peak ground accelerations and moment magnitudes used for preliminary liquefaction evaluations discussed in Section 4.3.3. Acceleration response spectra are provided by the Authority under separate cover.

4.3.3 Liquefaction

Liquefaction assessments for the CP2-3 alignment were performed for the OBE event using the subsurface data presented in the GDR. The analyses were conducted using peak ground accelerations of 0.08g and 0.09g for Fresno and Tulare Counties, respectively. A moment magnitude of 7.9 was used for all analyses. Utilizing the baseline groundwater levels, preliminary evaluations indicate soil liquefaction on a global basis is unlikely to occur during the OBE event on one of the nearby faults; however, localized liquefaction in discrete layers is possible.

For bidding purposes, assume liquefaction will not occur at the OBE; however, the Contractor is required to perform an independent liquefaction hazard analyses for final design.

4.4 Hydrogeologic Setting

4.4.1 Regional

The CP2-3 HSR alignment is located partially within the Kings, Kaweah, Tulare Lake, and Tule Subbasins. A hydrogeologic cross section of the basin is included in the CP2-3 GDR. Groundwater within these basins is managed by multiple stakeholders. Groundwater is the sole source of drinking water in the region. The current and potential uses of groundwater in the basin are municipal and domestic supply, industrial process supply, industrial service water supply, and agricultural and livestock water supply.

The regional groundwater flow direction in both Fresno and Tulare Counties is from east to west. There are some localized influences as a result of pumping, surface water treatment, and groundwater recharge appurtenances.

4.4.2 Major Aquifers

The depositional environment has formed a sequence of aquifers and aquitards that vary in thickness and lateral continuity. Aquifers are generally composed of granular water-bearing sediments, and aquitards are composed of finer-grained sediments that retard water flow. Three aquitards — A, B, and C — have been reported to exist at the north end of the CP2-3 alignment

(CH2M Hill 2005). Most of the aquifers underlying the study area are unconfined but can be semiconfined in isolated locations.

Generally, there are no extensive, low-permeability soils that isolate the upper aquifers from the lower aquifers. The Corcoran Clay (E-Clay) and correlative layers underlie the CP2-3 alignment from about the Kings River south at a depth of between 200 and 600 feet bgs (Saleeby 2003). These layers of clays are up to 120 feet thick and act as a local aquitard, but are not sufficiently regionally extensive to prevent groundwater flow.

4.4.3 Current Groundwater Conditions

Groundwater levels were monitored as part of the PE4P CP2-3 GI (refer to Section 3.2). The measured groundwater levels in southern Fresno County are deeper than historical levels while in Tulare County, the levels are shallower than anticipated. As described in the CP2-3 GDR (2014), the depth to current groundwater levels in Fresno County generally increases to the south and varies between 45 and 105 feet bgs. In Tulare County, in general groundwater levels vary between about 20 and 50 feet bgs. Where the proposed alignment crosses Deer Creek and the percolation ponds near Avenue 56, groundwater levels are shallower and vary between 10 and 20 feet bgs.

Groundwater measurements have not yet been performed in Kings County; however, the historical measurements performed by the California Department of Water Resources (2011) indicate the levels vary from about 20 to 100 feet (FB GSHR 2013).

Perched groundwater between 5 and 45 feet was encountered during the investigation in Tulare County and will likely be encountered during construction. The Soil Survey for Kings County prepared by the U.S. Department of Agriculture, Soil Conservation Service (USDA 1966) indicates that perched groundwater shallower than 6 feet may be present south of Cross Creek near Kansas Avenue.

Further discussion of perched groundwater conditions is included in Section 8.6. Baseline groundwater levels are presented in Section 6.2.

4.4.4 Land Subsidence

Many areas within the SJV have experienced significant subsidence due to groundwater extraction. The southern SJV has been the subject of an extensive investigation between 2007 and 2011 conducted by the Jet Propulsion Laboratories (JPL 2014) using remote sensing technology. The GDR includes the results of a cursory assessment of land subsidence made within the limits of CP2-3 by JPL. The JPL subsidence rate evaluation indicates that a significant subsidence bowl has developed between Hanford and Allensworth. The CP2-3 alignment nearly passes through the deepest part of this bowl. JPL has measured a subsidence rate of up to 25 cm/year (10 in/year) during the 3.5-year study period. Farther north and west of Hanford (not on the alignment studied for this report) along Highway 198, survey data and spot checks conducted in 2013 indicate the subsidence rates could be as high as 30.5 cm/year (12 in/year).

Section 5.0

Related Construction

5.0 Related Construction

The following is a brief description of several large, transportation-related infrastructure improvements in the vicinity of the proposed CP2-3 alignment from which some GI data have been obtained. These data provide some insight on large infrastructure construction in the vicinity of CP2-3. Four freeways of the California State Highway System either traverse or are adjacent to the proposed alignment. SR 43 and SR 198 traverses the alignment in Kings County while SR 43, SR 99 and SR 41 are either adjacent to or within about 2 miles of the alignment in Fresno County. The BNSF Railway is adjacent to the proposed alignment through much of Fresno and Tulare Counties, and the San Joaquin Valley Railroad crosses the alignment north of SR 198 in Kings County. The Stoil Spur extends to the west of the alignment south of Deer Creek in Tulare County.

In Fresno County, SR 41 runs parallel to the alignment about 2 miles to the west. This section of SR 41 is a four-lane divided highway constructed in the 1980s. It runs north–south and has overcrossing structures at W Clayton (Caltrans Bridge 42 0152) and W Lincoln Avenues (Caltrans Bridge 42 0144).

SR 99 is a four-lane divided highway. In Fresno County, it is about 1.5 to 2 miles east of the alignment. The nearest structure to the alignment is an overcrossing at E American Avenue (Caltrans Bridge 42 0205). California Highways and Public Works (1955, 1957a, 1957b, and 1960) provides background information on the construction of SR 99 through Fresno. The referenced articles do not contain technical and engineering information.

SR 43 (Central Valley Highway) is a two-lane rural highway that runs north–south through the SJV. The HSR alignment crosses SR 43 at four locations: north of Cole Slough in Fresno County, south of Jersey Avenue and north of the Lakeland Canal in Kings County, and south of Avenue 144 in Tulare County. SR 43 is adjacent to the HSR alignment and the BNSF Railway for about 2 miles in Kings County and about 10 miles in Tulare County. Several Caltrans structures have been constructed for SR 43 in the vicinity of the HSR alignment, including structures at Cole Slough (Caltrans Bridge 42 0081), Kings River (Caltrans Bridge 45 0064), Peoples Ditch (Caltrans Bridge 45 0061), SR 43 and SR 198 Separation (Caltrans Bridge 45 0080), East Branch Cross Creek (Caltrans Bridge 45 0053), Tule River (Caltrans Bridge 46 0122), Taylor Ditch (Caltrans Bridge 46 0123), Homeland Canal (Caltrans Bridge 46 0124), and Deer Creek (Caltrans Bridge 46 0238).

SR 198 is a four-lane divided expressway where it crosses the alignment east of Hanford in Kings County. It was initially constructed as a two-lane county highway in the 1910s and expanded to the current four-lane expressway in 2012. It has two major structures in the vicinity of the HSR alignment: a bridge over the Lakeside Canal (Caltrans Bridge 45 0005) and the SR 43 and SR 198 separation (Caltrans Bridge 45 0080).

Geotechnical logs of test borings and as-built drawings for several overpasses and bridges along these freeways were collected from a Caltrans database. These logs of test borings are presented in Appendix A of the FB CP2-3 GDR.

Additional information regarding construction methods, ground behavior, groundwater conditions, ground support methods, and problems during construction was not provided in the as-built construction records obtained from Caltrans.

Information from the adjacent railroads was not obtained during this and previous design phases of the project.

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Section 6.0

Ground Characterization

6.0 Ground Characterization

6.1 Baseline Description of Subsurface Conditions

Subsurface soils have been characterized separately for the alignment in Fresno County and Tulare County. These political boundaries provide a convenient basis for evaluating ground conditions because the portions of alignment within Fresno and Tulare Counties lie in different dominant geologies, separated by approximately 27 miles of Kings County. The following sections present the development of baseline ground conditions in each county for materials of similar character and engineering properties. Baseline engineering parameters associated with these ground conditions are presented later in Section 6.5.

Several trends were observed in the soils data when grouped by depth and by location along the alignment. Histograms have been used to assist the development of baselines that often reference the percentage of a particular soil type, which was calculated by dividing by the total depth drilled. For instance, to calculate the percentage of SM soil below 35 feet bgs in a 100-foot borehole, the cumulative length drilled through SM soil was divided by 65 feet.

Near-surface material composed of Existing Fill or shallow Native Soils is baselined first, for both Fresno and Tulare Counties combined.

Fresno County Native Soils are divided by depth to group similar engineering properties above and below 25 feet bgs. Hardpan soils are included in the Native Soils discussions of Fresno County (Sections 6.1.3 and 6.5.2.1).

Tulare County Native Soils require more divisions to identify trends in the engineering parameters because they are interbedded, varying in plasticity and density, and occasionally include perched or locally elevated groundwater conditions. Baseline ground conditions in Tulare County are divided by depth (35 feet bgs) and soil type (‘coarse’ and ‘fine’), as well as longitudinally along the alignment.

As explained in Section 1.0, there are no recent geotechnical data available for the portion of alignment within Kings County.

6.1.1 Subsurface Conditions in Fresno and Tulare Counties

A histogram of United Soil Classification System (USCS) soil types, as encountered in boreholes and classified during investigation, supported by laboratory testing, is shown in Figure 6.1-1. The distribution of soil types for Fresno County is shown in yellow, and the distribution for Tulare County is shown in orange.

The histogram illustrates the nature of the soils native to Fresno County — predominantly alluvial fan deposits comprising interbedded layers of poorly graded sand, sandy silt, and silty sand. By comparison, the predominantly lacustrine deposits of Tulare County contain a greater proportion of fines, encountered as interbedded sand, silt, and clay.

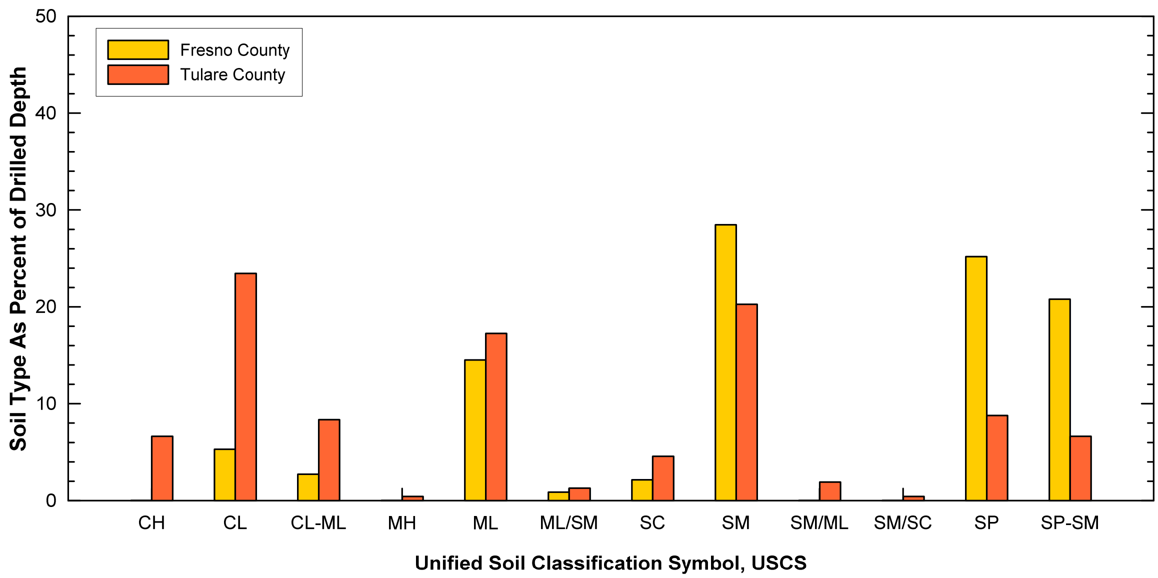

Figure 6.1-1
USCS Distribution by County

A histogram of normalized CPT soil behavior type (SBT_N) is shown in Figure 6.1-2. The SBT_N classification was unified by Robertson (2010), with earlier SBT numbering from Robertson (1990), and may be used as a guide to predict soil behavior based on the data collected during cone penetration.

Figure 6.1-2 shows that the SBT encountered in Fresno County are predominantly of SBT_N 5 and 6, with a lesser prevalence of soils of SBT_N 3 and 4. The SBTs encountered in Tulare County are predominately SBT_N 3 and 4.

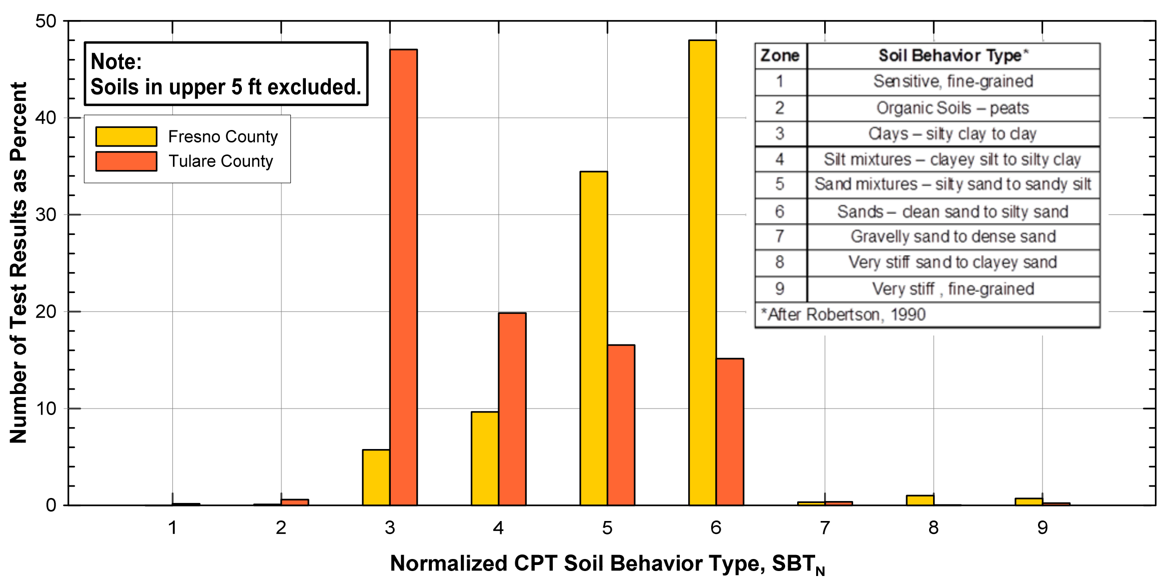

Figure 6.1-2
SBT_N Distribution for by County

Existing Fill material overlies the natural soils of both counties, and this stratum is discussed in Section 6.1.2. The soils native to Fresno County and Tulare County are discussed in Sections 6.1.3 and 6.1.4, respectively.

6.1.2 Existing Fill

Existing Fill encountered during the GI varied from less than 1 to 13 feet in thickness. The depth of Existing Fill was identified primarily from hand augering during utility clearance prior to the drilling of boreholes. Existing Fill was encountered in 12 of the 19 PE4P boreholes along the CP2-3 alignment. A fill layer was not explicitly identified in seven of the borehole logs, and in these locations the extent of Fill has been reported as less than 1 foot.

In Fresno County, Existing Fill was encountered in seven of nine boreholes and predominantly consisted of sand (SP) and silty sand (SM). In Tulare County, Existing Fill was encountered in 5 of 10 boreholes and consisted of more variable sand (SP) and silty sand (SM), with greater fines and layers of clayey sand (SC), sandy clay (CL), and sandy silt (ML).

Existing Fill was typically coarser in Fresno County than in Tulare County, as indicated by the histogram in Figure 6.1-3. This is consistent with the character of the Native Soils of each county and to an extent expected, as fill material is often derived from local borrow areas. The borehole-by-borehole breakdown of USCS soil type is presented in Table 6.1-1, and the grain size distributions for eight samples of sandy fill are presented in Figure 6.1-4.

It is important to note the relatively small sample size of tests and the variable nature of fill material in general.

As a baseline, assume that 95% in situ volume of Existing Fill in Fresno County is coarse-grained (SM, SP, or SP-SM) and 5% is predominantly fine-grained (CL or ML). As a baseline for Tulare County, assume that 60% in situ volume of Existing Fill is coarse-grained (SM, SP, or SP-SM), and 40% is predominantly fine-grained (CL or ML).

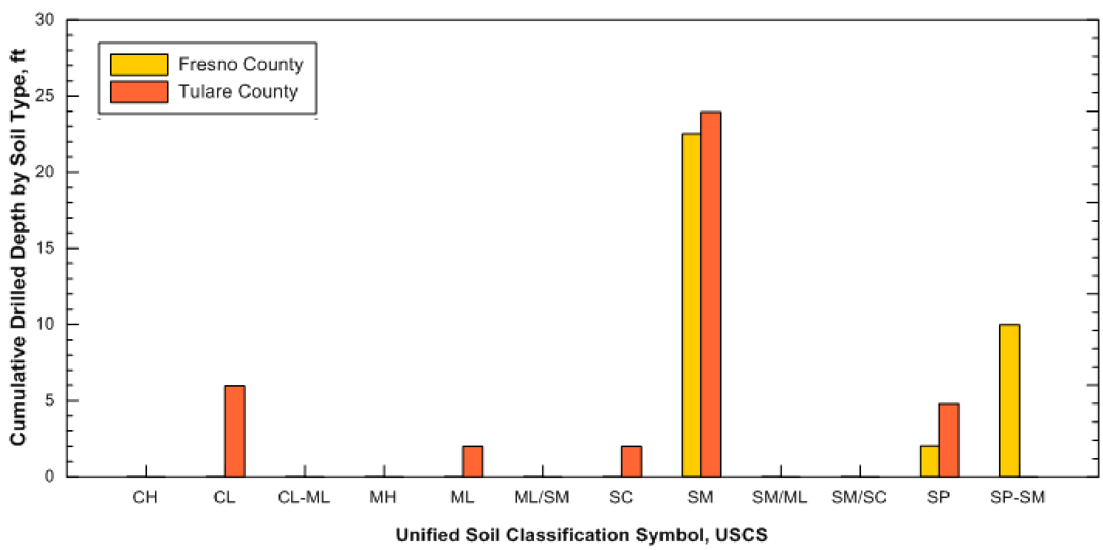

Figure 6.1-3
USCS Distribution of Existing Fill

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

CALIFORNIA High-Speed Rail Authority

Table 6.1-1
USCS Distribution of Existing Fill by Percentage of Depth Explored

Borehole ID	Fill Depth (ft)	AC (%)	CL (%)	ML (%)	SC (%)	SM (%)	SP (%)	SP- SM (%)
S0019AR[F]	**7.5**	0	0	0	0	100	0	0
S0020R[F]	**2**	0	0	0	0	0	100	0
S0021R[F]	**8**	0	0	0	0	100	0	0
S0028R[F]	**5**	0.8	0	0	0	0	0	99.2
S0029R[F]	**≤1**	–	–	–	–	–	–	–
S0030R[F]	**4**	0	0	0	0	100	0	0
S0031R[F]	**5**	0	0	0	0	0	0	100
S0033AR[F]	**3**	0	0	0	0	100	0	0
S0034BR[F]	**≤1**	–	–	–	–	–	–	–
S0065R[T]	**6.2**	0	0	0	32	68	0	0
S0066R[T]	**13**	0	0	15	0	85	0	0
S0067R[T]	**10**	0	52	0	0	0	48	0
S0068R[T]	**3**	0	0	0	0	100	0	0
S0069R[T]	**≤1**	–	–	–	–	–	–	–
S0069AR[T]	**≤1**	–	–	–	–	–	–	–
S0070R[T]	**≤1**	–	–	–	–	–	–	–
S0071R[T]	**≤1**	–	–	–	–	–	–	–
S0072R[T]	**6.5**	0	12	0	0	88	0	0
S0073R[T]	**≤1**	–	–	–	–	–	–	–

[F] Borehole located in Fresno County
[T] Borehole located in Tulare County

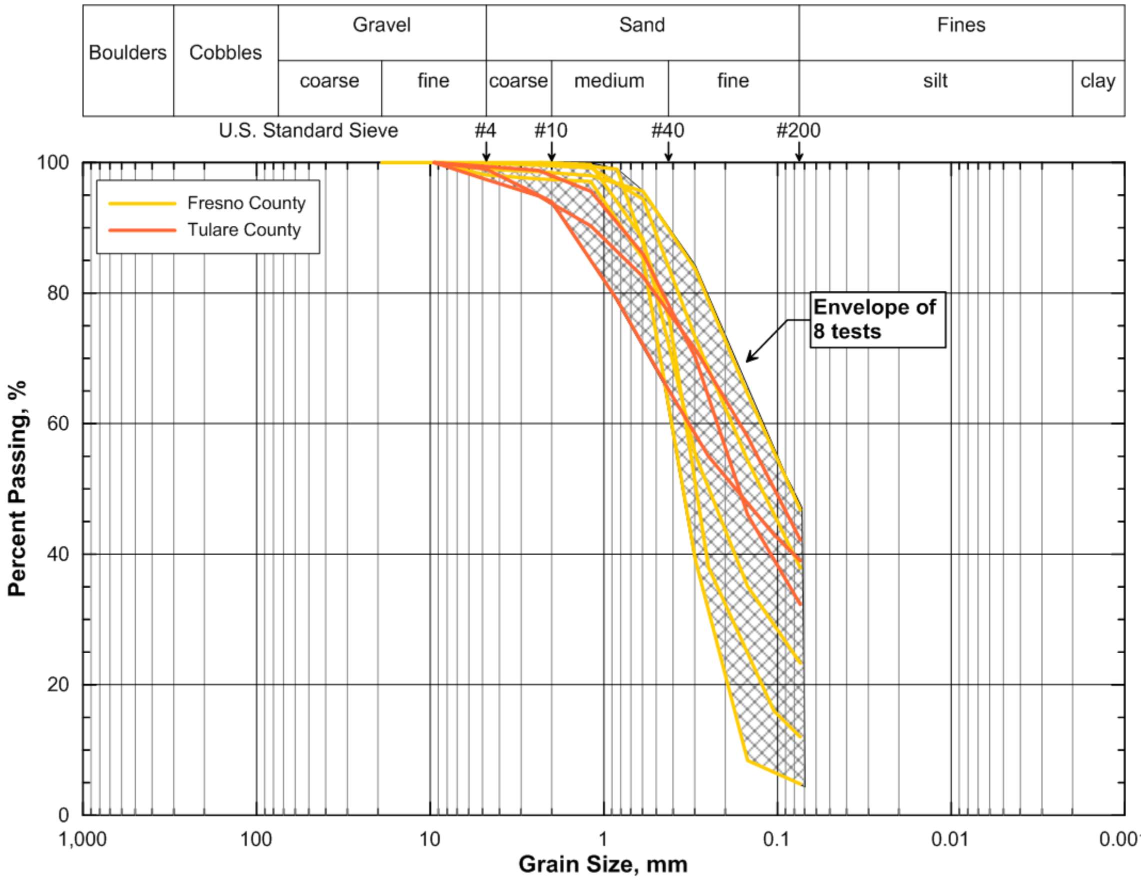

Figure 6.1-4
Representative Grain Size Distribution of Existing Fill Encountered in Fresno and Tulare Counties

Existing Fill can also include surface pavements consisting of asphalt concrete (AC), concrete, and aggregate base. Broken asphalt was encountered in S0028R and was a half inch thick.

For accessibility, GI was often undertaken adjacent to existing roadways and therefore may not coincide with the actual alignment and accurately reflect depth of fill or proportion of existing or abandoned pavements requiring excavation. Therefore, areas with deeper Existing Fill or with Existing Fill containing variable debris are likely to be present between exploratory holes. In the Fresno Area, it is not uncommon to encounter debris of unknown origin during construction excavations.

In general, the majority of the alignment is expected to encounter Existing Fill. The greatest depths of Existing Fill can be expected where the alignment crosses existing roadway embankments and where the alignment is in close proximity to crossings and grade separations. No historical records describing how Existing Fill was placed and compacted were located during preliminary desk studies.

The nature of drilling and sampling methods used and borehole spacing makes it difficult to quantify the maximum size of fragments in Existing Fill. For bidding purposes, assume debris up to 1 foot in greatest dimension is present in Existing Fill. Debris most commonly pertains to rock fragments, but may also include rubbish, rubble, or remnants of previous development.

Where present (based on the design-builder's review of Reference Drawings and Existing Conditions), assume existing AC or concrete pavements are 3 inches thick on minor roads and 8 inches thick on improved sections of SR 43 or other highways. For bidding purposes, assume minor roads have 6 inches of gravel or aggregate base underlying the AC, and the highways have 12 inches of aggregate base underlying the AC. Do not assume the existing aggregate base can be directly reused as aggregate base.

Insufficient data are available to develop baseline parameters of soil laden with organics or disturbed from previous site uses (such as farm fields, orchards, or existing development).

6.1.3 Native Soils of Fresno County

The Native Soils underlying Existing Fill in Fresno County are predominantly associated with the Alluvial Fan geologic unit (Qc, Qf, Qs), consisting of interbedded layers of poorly graded sand and silt, with varying amounts of coarse- and fine-grained particles. Interlayers of these units are classified as sand (SP), sand with silt (SP-SM), silty sand (SM), clay (CL), clayey silt (CL-ML), and sandy silt to silt (ML).

The distribution of USCS soil type by borehole is provided in Table 6.1-2. The predominant soils encountered in all boreholes include sand and silty sand.

Table 6.1-2
Distribution for Native Soils Encountered in Fresno County by Percentage of Depth Explored

Borehole ID	Depth[a] (ft)	SP	SP-SM	SM	SC	SM/ML[b]	ML	CL-ML	CL
S0019AR	74	37.2	17.2	35.8	0.0	0.0	9.8	0.0	0
S0020R	99.5	26.2	10.1	24.2	0.0	0.0	39.5	0.0	0
S0021R	78.5	22.3	46.5	19.1	0.0	6.4	5.7	0.0	0
S0028R	160	23.4	15.5	37.6	3.8	0.0	9.6	0.0	10.2
S0029R	125	17.6	14.4	26.9	0.0	0.0	16.6	8.0	16.6
S0030R	97.5	13.8	16.2	42.6	8.2	0.0	9.2	6.1	3.8
S0031R	76.5	35.3	38.4	16.3	0.7	3.9	0.0	4.4	0.9
S0033AR	98.5	34.0	26.4	19.3	0.0	0.0	15.2	0.0	5.1
S0034BR	100.8	24.5	15.9	26.7	4.9	0.0	20.8	5.5	1.8

[a] Depth of borehole below Existing Fill (i.e., exploration depth in Native Soils)
[b] Borderline classification at boundary between fine-grained and coarse-grained soil, sandy silt/silty sand

The proportion of fines within the sands and the frequency of finer-grained layers within the alluvial sequence tend to increase with depth. This transition is most distinct at a depth of approximately 25 feet, and is illustrated in the histogram for USCS distribution in Figure 6.1-5, but more notably in the histogram for SBT_N distribution in Figure 6.1-6.

Hardpan soils were encountered in some areas of Fresno County but not consistently or to great depths; refer to Section 6.5.2.1 for further discussion.

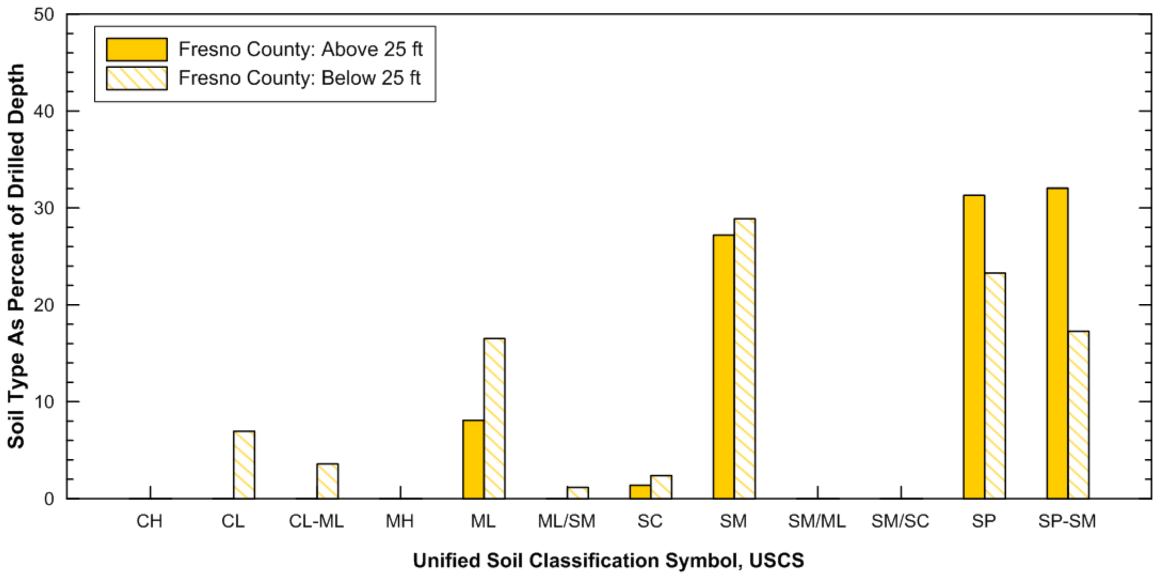

Figure 6.1-5
USCS Distribution for Native Soil Encountered in Fresno County

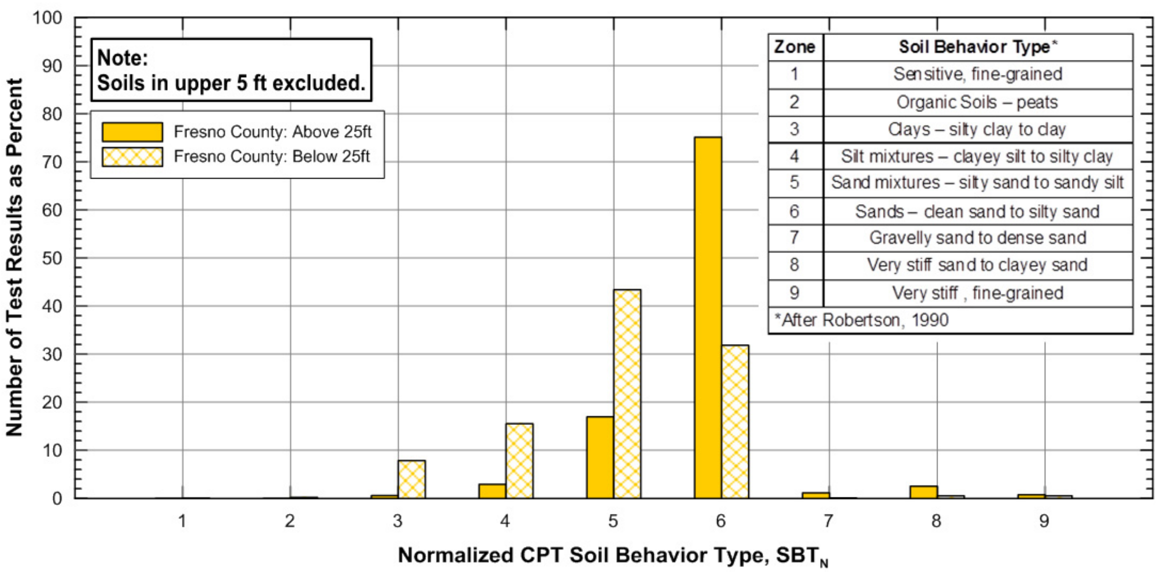

Figure 6.1-6
SBT$_N$ Distribution for Native Soil Encountered in Fresno County

The grain size distribution curves for Native Soil of Fresno County further support the trend of finer grained soil below 25 feet bgs, and are presented in Figure 6.1-7. These curves represent the results of laboratory sieve and hydrometer testing performed on samples of soil from boreholes drilled during the PE4P investigation. The frequency of gradation tests with depth are shown in Figure 6.1-8.

For coarser particles, only two samples in borehole S0034BR encountered gravel greater than 3% by weight. Samples from 30 feet and 45 feet bgs contained 15% and 21% gravel, respectively.

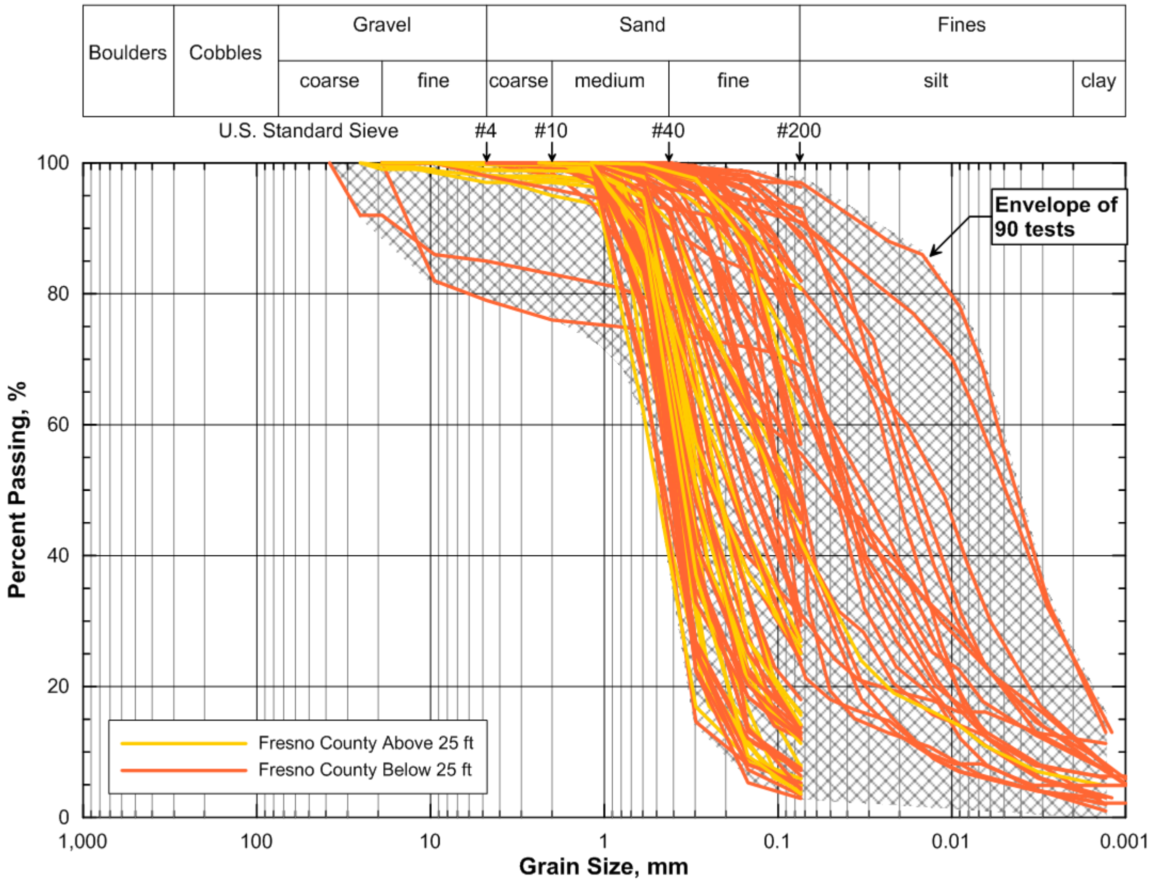

Figure 6.1-7
Grain Size Distribution of Native Soils Encountered in Fresno County

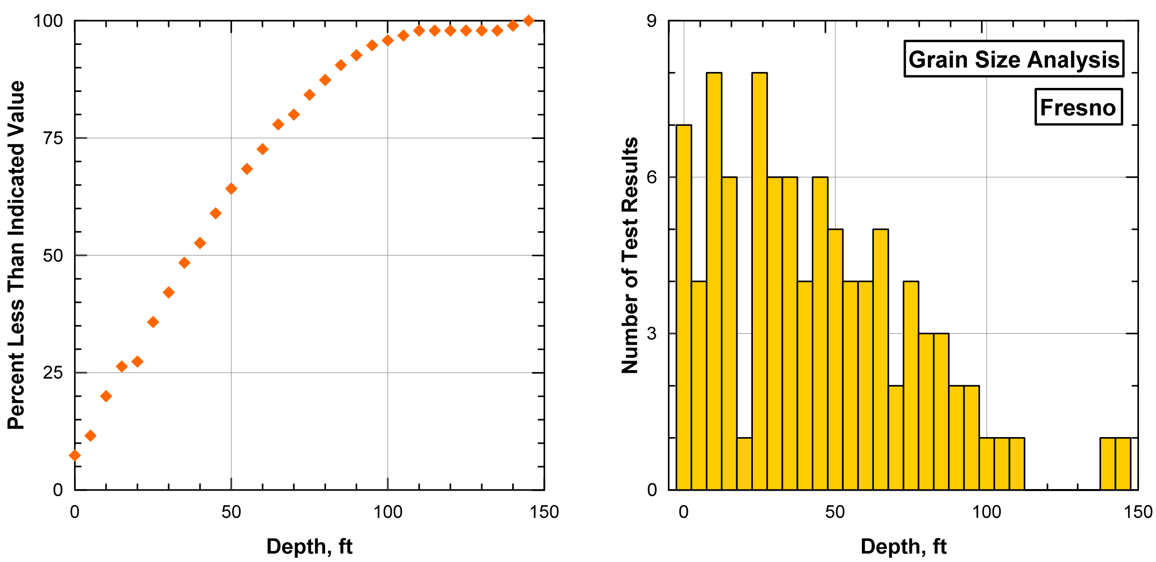

Figure 6.1-8
Probability Distribution and Frequency of Grain Size Analysis with Depth Encountered in Fresno County

Atterberg Limits tests were carried out on nine samples, which reflects the limited amount of fine grained material encountered in the Fresno County boreholes. The distribution of plasticity results are presented in Figure 6.1-9. All tested materials were inorganic and plotted with USCS identifications of clay (CL) and clayey silt (CL-ML) to silt (ML).

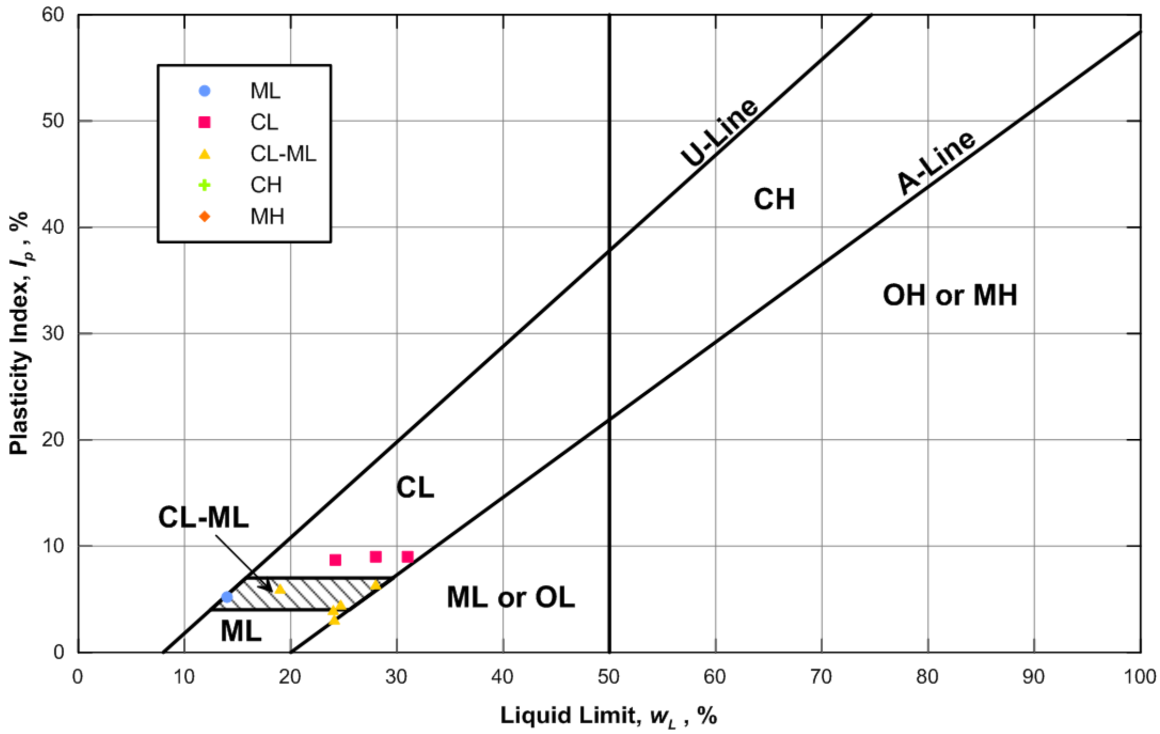

Figure 6.1-9
Plasticity Characteristics of Native Soils Encountered in Fresno County

Assessment provided in the GSHR suggests the potential for Dune Sands (Qsd) to be present in the northern portion of the CP2-3 alignment in Fresno County. Dune Sands are windblown deposits likely to overlie the more prevalent Alluvial Fan deposits, where present. Dune Sand is generally poorly graded and loosely compacted, and thus may be subject to higher settlements and hydrocompaction.

In general, the borehole logs indicate soils representative of Alluvial Fan deposits. Identification of Dune Sand during the PE4P investigation was complicated by predrilling of the uppermost 5 feet of ground. Although no SPTs were taken in the upper 5 feet, borehole logs indicate near surface soils are loose and generally fine or fine to medium sand, often denoted as Existing Fill. Thus, it is possible that Dune Sand is present and variable across the Fresno County portions of the CP2-3 alignment. Identification of Dune Sand and quantification of any associated risks shall be a target outcome of the design-builder's GI. Assumptions for bidding purposes are discussed in Section 6.6.6.

6.1.4 Native Soils of Tulare County

6.1.4.1 Tulare County Soil Types

The Native Soils underlying Existing Fill in the areas explored in Tulare County are predominantly associated with Lacustrine Deposits (Ql), consisting of interbedded layers of sand, silt, and clay. Interlayers of this unit are classified as sand (SP), sand with silt (SP-SM), silty sand (SM), sand

with clay (SC), sand/silt (SM/ML), sandy silt to silt (ML), elastic silt with sand (MH), clayey silt to silty clay (CL-ML), sandy lean clay to lean clay (CL), and sandy fat clay to fat clay (CH). The coarse-grained soils are generally medium dense to very dense, and the fine-grained soils are generally stiff to hard.

The distribution of USCS soil type by borehole is provided in Table 6.1-3. The methodology used to develop percentages by soil type is the same as was introduced in Section 6.1. For simplicity, these percentages have been summed into more general coarse grained and fine grained categories, as defined in Table 6.1-3 and further discussed later in this section.

There is no predominant soil classification type in the Native Soils of Tulare County, which alternate between layers of coarse grained and fine grained material. Approximately 45% of all native soils drilled in Tulare County are classified as coarse-grained and 55% as fine-grained.

Table 6.1-3
USCS Distribution of Native Soils in Tulare County by Boring and Percentage of Depth Explored

Borehole ID	Depth[a] (ft)	Coarse-Grained %						Fine-Grained %					
		Total	SP	SP-SM	SM	SC	SM/ML	Total	ML	MH	CL-ML	CL	CH
S0065R	**95.3**	**56.6**	16.3	19.4	21.0	0.0	0.0	**43.4**	17.6	0.0	6.9	18.9	0.0
S0066R	**88.5**	**74.6**	5.6	11.9	57.1	0.0	0.0	**25.4**	4.2	0.0	5.6	15.6	0.0
S0067R	**141.5**	**63.9**	31.0	3.9	12.7	16.3	0.0	**36.1**	8.1	0.0	0.4	27.7	0.0
S0068R	**148.5**	**41.3**	0.0	10.9	18.3	0.0	12.1	**58.7**	32.5	0.0	9.9	16.3	0.0
S0069R	**101.5**	**39.1**	0.0	4.4	32.2	0.0	2.5	**60.9**	9.4	0.0	6.6	41.0	3.9
S0069AR	**101.5**	**34.0**	18.2	0.0	15.8	0.0	0.0	**66.0**	12.8	0.0	7.6	45.6	0.0
S0070R	**101.5**	**39.7**	4.9	0.0	19.1	15.7	0.0	**60.3**	22.2	0.0	13.8	24.3	0.0
S0071R	**151.5**	**39.8**	6.3	11.5	10.7	3.3	7.9	**60.2**	11.1	0.0	0.0	26.3	22.8
S0072R	**158.5**	**26.9**	3.3	3.2	14.4	2.8	3.2	**73.1**	17.9	0.0	18.1	15.5	21.6
S0073R	**81.5**	**29.7**	0.0	0.0	17.5	12.3	0.0	**70.3**	38.7	6.1	16.9	2.8	5.8
Average		**44.5**	*8.6*	*6.5*	*21.9*	*5.0*	*2.6*	***55.5***	*17.5*	*0.6*	*8.6*	*23.4*	*5.4*
Min		**26.9**	*0.0*	*0.0*	*10.7*	*0.0*	*0.0*	***25.4***	*4.2*	*0.0*	*0.0*	*2.8*	*0.0*
Max		**74.6**	*31.0*	*19.4*	*57.1*	*16.3*	*12.1*	***73.1***	*38.7*	*6.1*	*18.1*	*45.6*	*22.8*
Stdev		**15.5**	–	–	–	–	–	***15.5***	–	–	–	–	–

[a] Depth of borehole below Existing Fill (i.e., exploration depth in Native Soils)

6.1.4.2 Tulare County Soil Type Trends

While there is no compelling overall trend with respect to soil type distribution with depth across Tulare County, an apparent increase in density and consistency was detected at approximately 35 feet bgs. This is evident in the parameters introduced in Section 6.5.3.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

The distribution of soil by type above and below 35 feet bgs is illustrated in the histogram for USCS distribution in Figure 6.1-10 and the histogram for SBT_N distribution in Figure 6.1-11. The USCS distribution indicates no trend, but the SBT_N distribution is suggestive of greater fines below 35 feet bgs.

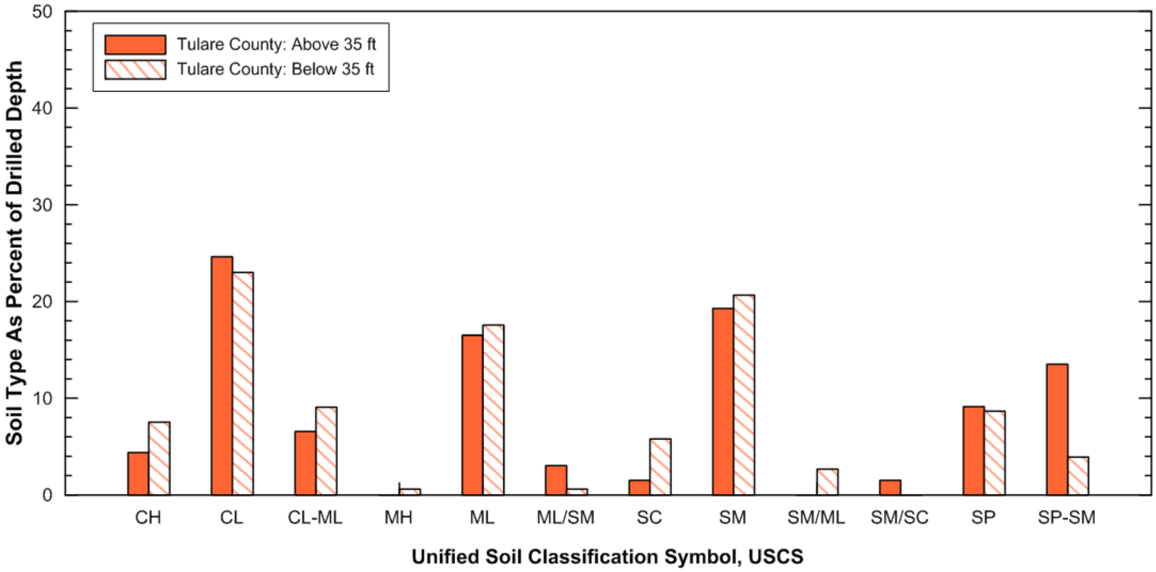

Figure 6.1-10
USCS Distribution for Native Soil in Tulare County

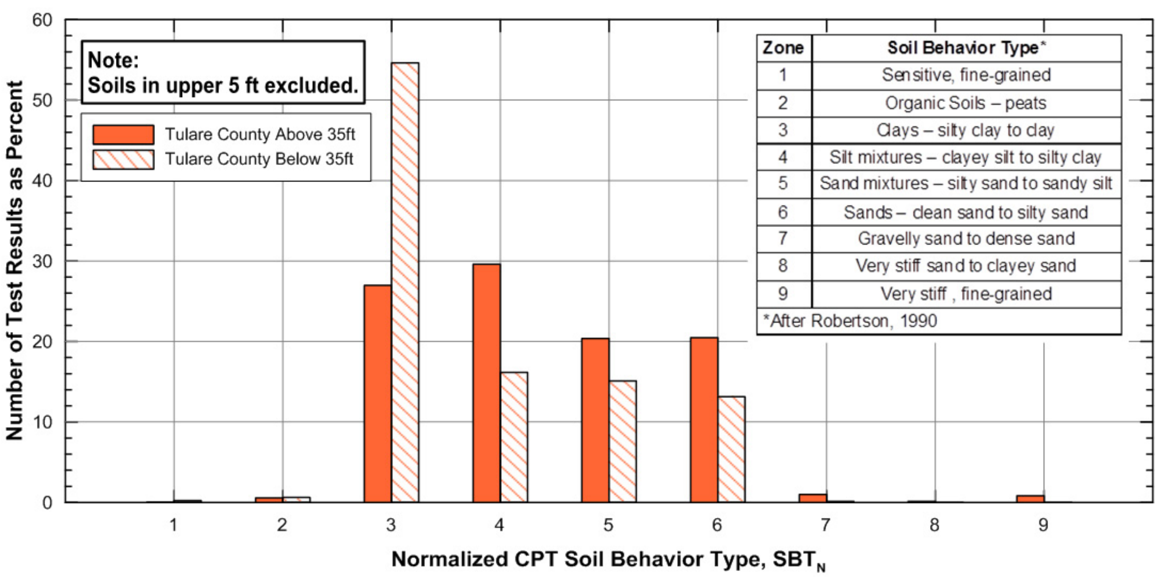

Figure 6.1-11
SBT_N Distribution for Native Soil in Tulare County

Grain size distribution curves for Native Soil of Tulare County are presented in Figure 6.1-12. These curves represent the results of laboratory sieve and hydrometer testing performed on samples of soil from boreholes drilled during the PE4P investigation. The frequency of gradation tests with depth are shown in Figure 6.1-13. The results suggest a greater proportion of fines in samples tested from below 35 feet bgs.

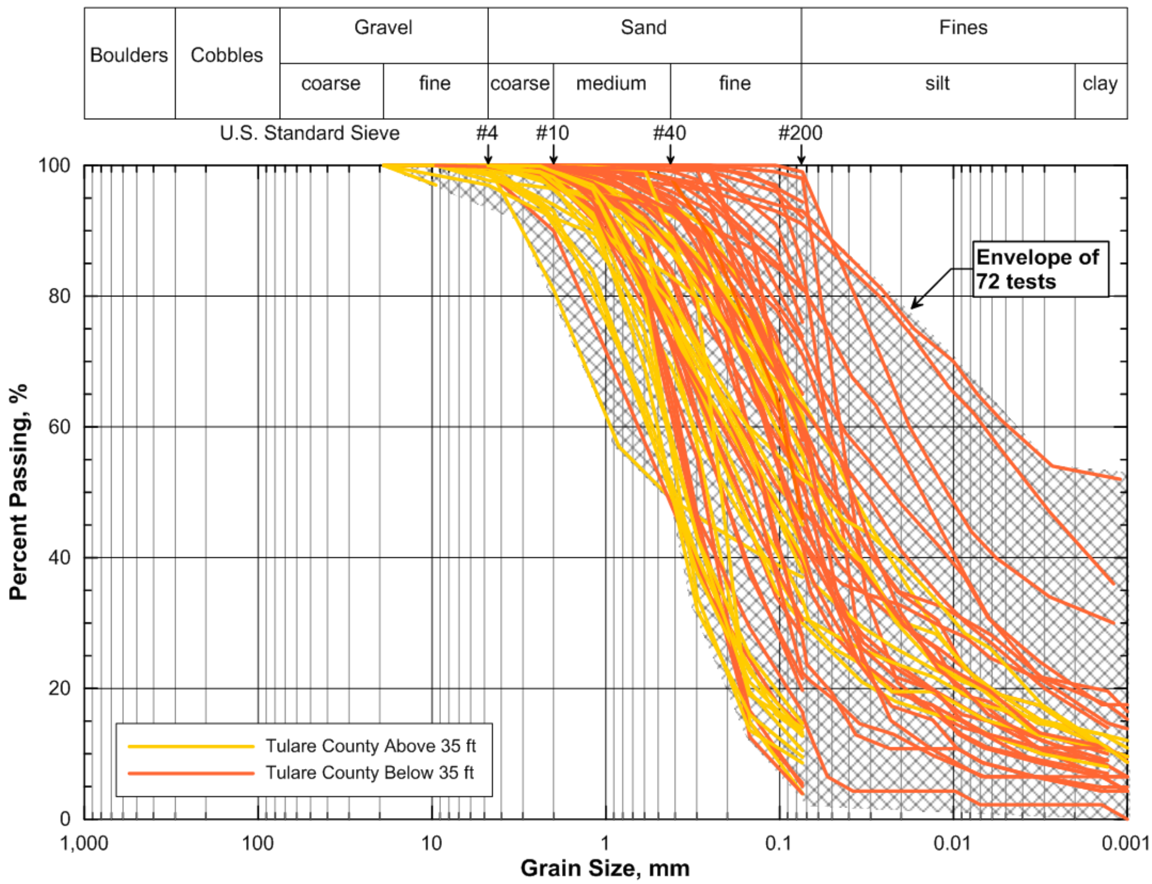

Figure 6.1-12
Grain Size Distribution of Native Soils Encountered in Tulare County

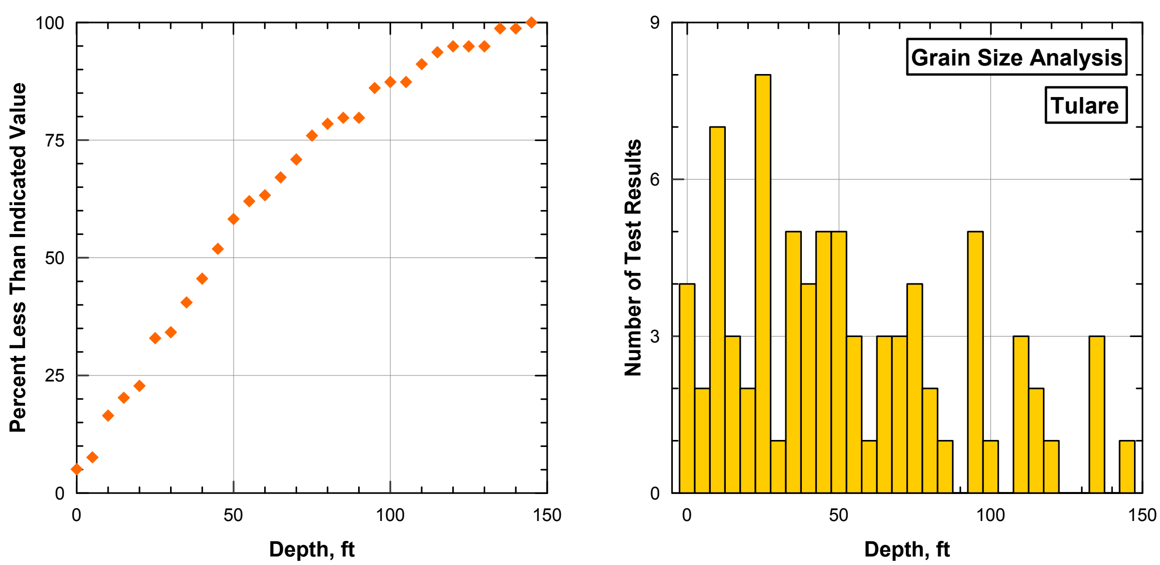

Figure 6.1-13
Probability Distribution and Frequency of Grain Size Analysis with Depth in Tulare County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Atterberg Limits tests were carried out on 44 samples, and the distribution of plasticity results are presented in Figure 6.1-14. The probability distribution of Atterberg limits tests with depth is presented in Figure 6.1-15. All tested materials were inorganic and plotted with USCS identifications of predominantly clay (CL) and clayey silt (CL-ML) to silt (ML). High-plasticity clay (CH) samples appear with greater frequency in boreholes S0071R, S0072R, and S0073R, located south of Deer Creek, over the final 8 miles of the CP2-3 alignment. High-plasticity clay was also encountered in borehole S0069R at shallow depth.

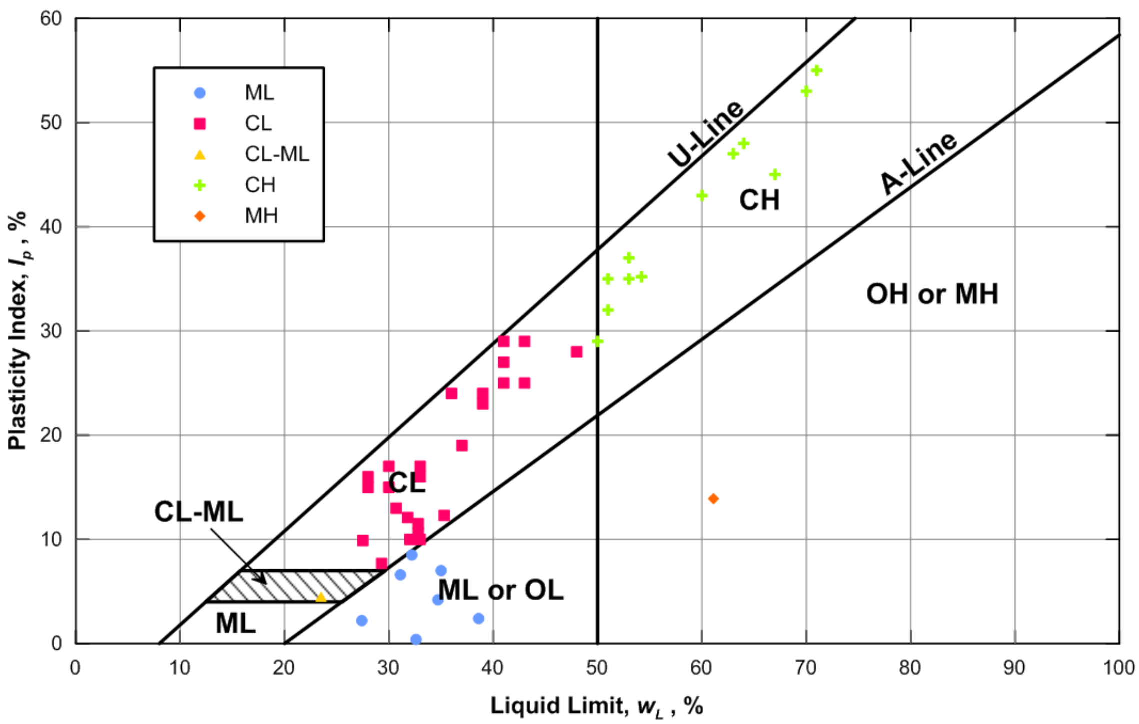

Figure 6.1-14
Plasticity Characteristics of Native Soils Encountered in Tulare County

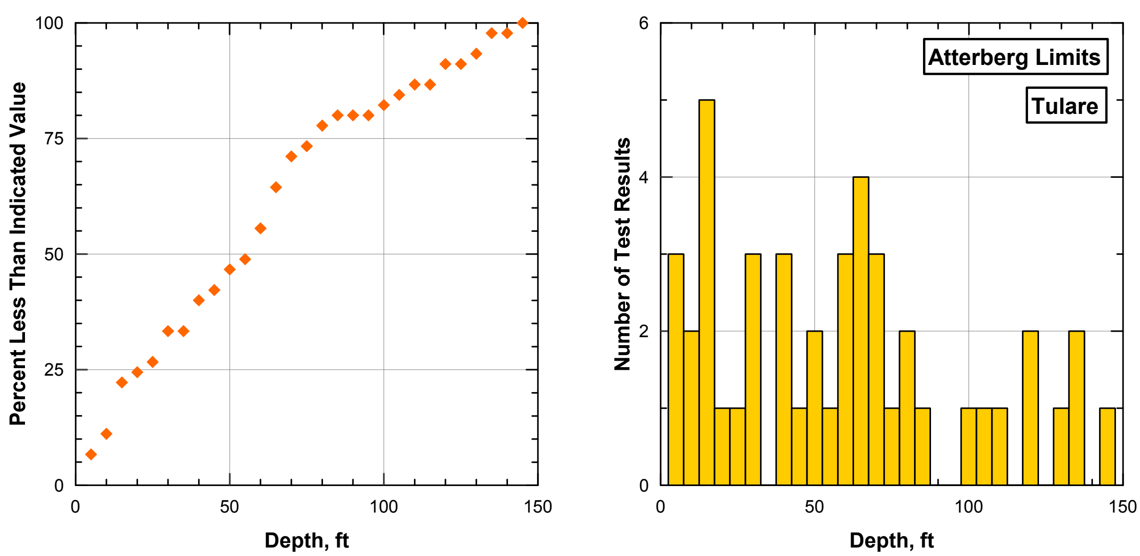

Figure 6.1-15
Probability Distribution and Frequency of Atterberg Limits Tests with Depth in Tulare County

For purposes of ground characterization, it is convenient to cluster soil types into coarse-grained and fine-grained categories that rely primarily on percent fines and expectations of engineering behavior. For example, fine-grained materials are categorized based on the potential to exhibit a discernible undrained response during transient loading. The definitions provided in Table 6.1-4 have been adopted to facilitate further data analysis and enable the baselining of engineering properties in the highly stratified geologic conditions of Tulare County.

Table 6.1-4
Definition of Coarse-Grained and Fine-Grained Categories for Native Soils Encountered in Tulare County

Category	Source of Material Identification	Material Types
Coarse-Grained	Borehole Samples by USCS	SP, SP-SM, SM, SC, SM/ML [b]
	CPT data by SBT_N [a]	5, 6, 7, and 8
Fine-Grained	Borehole Samples by USCS	ML, MH, CL-ML, CL, CH
	CPT data by SBT_N [a]	1, 2, 3, 4, and 9

[a] SBT_N after Robertson 1990.

[b] Borderline classifications at the boundary between fine- and coarse-grained soil, includes sandy silt (SM/ML) and silty sand (ML/SM). This material has been included in the coarse-grained category as it is likely to behave in a drained manner when subject to short term loading and therefore is more relevant to engineering parameters for coarse-grained soils.

The coarse and fine criteria above have been used to evaluate the stratigraphic variation in the boreholes and CPTs of Tulare County. The following paragraphs describe the results of evaluations of the interbedding as evidenced in the available data.

Boreholes are sampled typically at 5-foot intervals with samplers ranging in length from 1.5 feet to 3 feet. This convention results in soils between samples not being identified, and the interpolations between samples made on borehole logs may not be representative of actual conditions. Individual coarse- and fine-grained layers, as inferred from the boreholes, range in thickness from 0.5 feet to 47.5 feet. The average layer thickness, above or below 35 feet, is approximately 10 feet.

CPTs yield near-continuous data with readings taken every 2 inches of penetration. While physical samples were not obtained during the CPT pushes, side-by-side calibration of CPT data interpretations with the results of an adjacent borehole can allow for SBT_N from the CPT measurements to be compared to the USCS soil type. Well-calibrated SBT_N interpretations can be used to estimate stratigraphic changes where borings have not been completed. Comparison of S0072R and S0220CPT supports the conclusion that correlation with SBT_N provides good agreement with the borehole results. On this basis, CPT SBT_N has been used to investigate the spatial distribution of coarse and fine layers in the Native Soil of Tulare County.

The percentage of soil type by SBT_N is presented by CPT location for above and below 35 feet bgs in Figure 6.1-16 and Figure 6.1-17, respectively. For reference, the increasing CPT numbering (reading the figures from left to right) follows the alignment from north to south. A detailed presentation of the methodology and results of this assessment is provided in Appendix A.

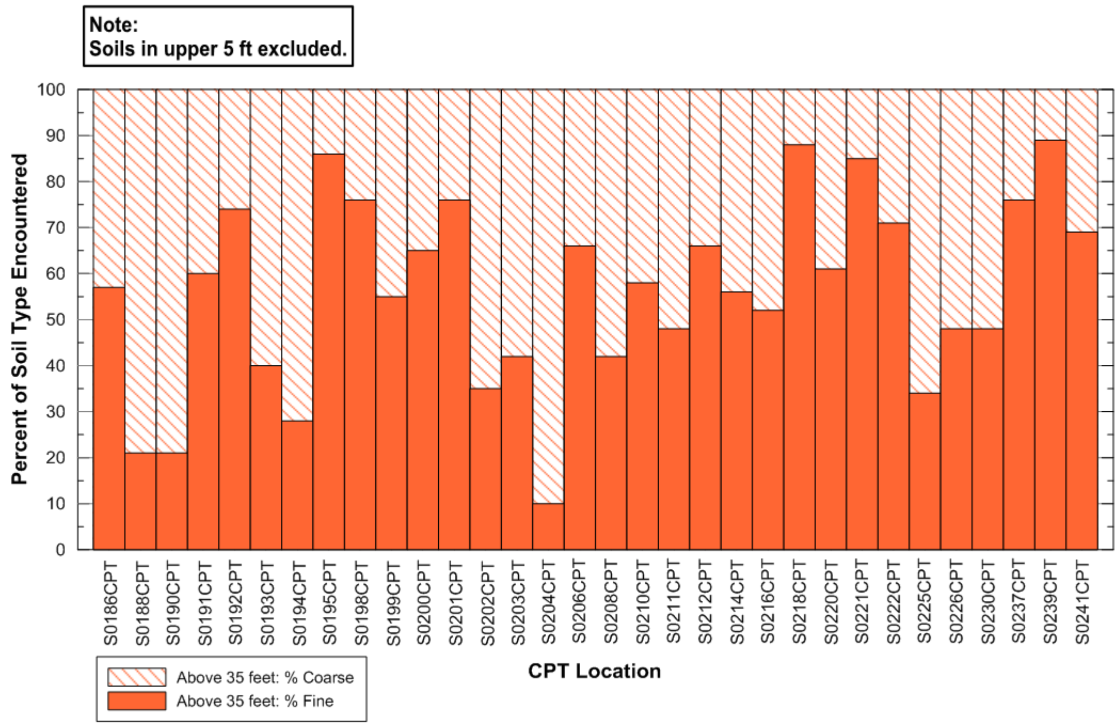

Figure 6.1-16
Percentage of Soil Type Above 35 Feet bgs by CPT Location in Tulare County

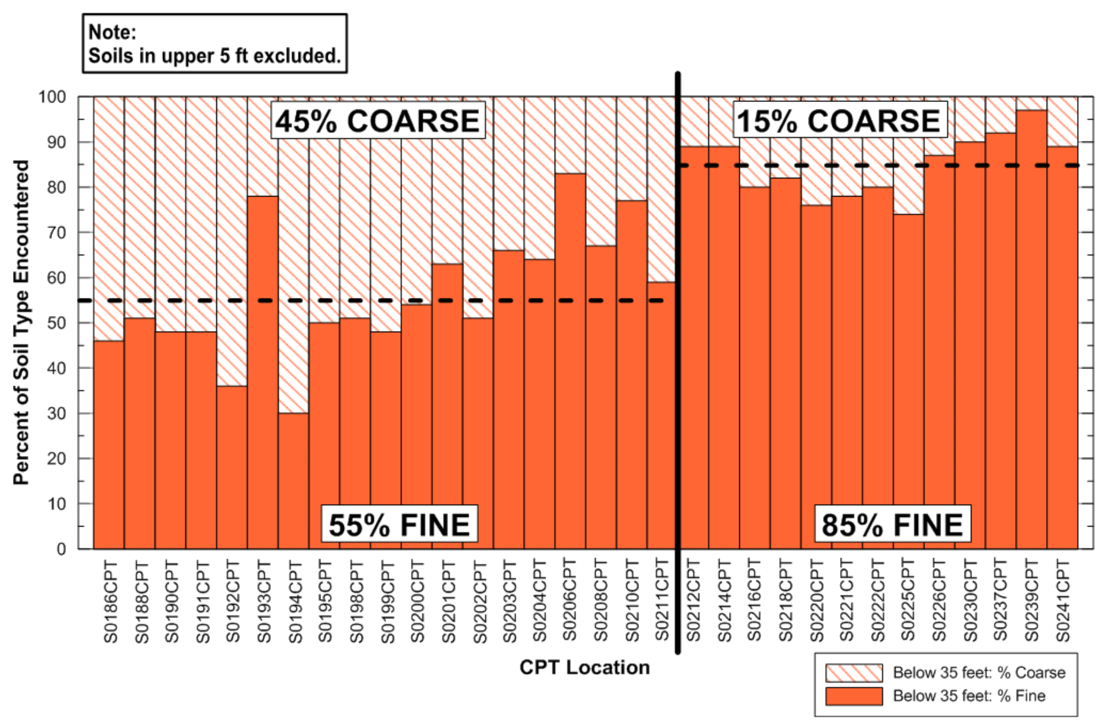

Figure 6.1-17
Percentage of Soil Type Below 35 Feet bgs by CPT Location in Tulare County

Above 35 feet bgs, the distribution of coarse and fine grained soil is variable and no clear spatial trends are apparent. The results vary significantly, from 90%/10% coarse/fine in S0204CPT to 11%/89% coarse/fine in S0239CPT.

As a baseline for design parameters in native soils above 35 feet bgs, assume that 60% of soil is coarse-grained and 40% of soil is fine-grained. This approximately represents a skew of one standard deviation in favor of coarse-grained soil.

Below 35 feet bgs, the distributions of coarse and fine grained soil varies from 70%/30% coarse/fine in S0194CPT to 3%/97% coarse/fine in S0239CPT. However, a distinct trend of increasing fines below 35 feet is observed southward beginning with S0203CPT and becoming more significant and consistent from S0212CPT, as shown in Figure 6.1-17. S0212CPT coincides with Avenue 88.

A point 500 feet north of Avenue 88 can be used as an approximate division for the baseline parameters. As a baseline for native soils above 35 feet bgs and north of that point, assume that 45% of soil is coarse-grained and 55% of soil is fine-grained. As a baseline for native soils above 35 feet bgs and south of that point, assume that 15% of soil is coarse-grained and 85% of soil is fine-grained.

The thickness of individual coarse and fine interlayers encountered varies widely. An average layer thickness of 5 feet is indicated by the CPT results.

6.2 Groundwater Level

For design of permanent structures in Fresno County, assume a baseline groundwater depth of 40 feet bgs. Groundwater during construction periods could be significantly deeper, indicated in the CP2-3 GDR to vary from 45 to 105 feet bgs.

In Tulare County, the groundwater table varies by location. For construction conditions, assume a baseline depth to groundwater as follows:

- 40 feet bgs north of Avenue 84.
- 10 feet bgs between Avenue 84 and Avenue 68 (vicinity of Deer Creek).
- 20 feet bgs south of Avenue 68.

For design of permanent structures in Tulare County assume a baseline groundwater table 30 feet bgs north of Avenue 84 and 10 feet bgs south of Avenue 84.

Potential for perched groundwater is discussed in Section 8.6.

6.3 Contaminated Soil

The GI did not encounter or test for contaminated material; however, it may exist within the CP2-3 project area. No baseline for contamination is provided herein.

6.4 Corrosivity

6.4.1 Soil Chemistry

Corrosion tests were performed on five representative soil samples to evaluate the corrosion potential for buried iron, steel, mortar-coated steel, and reinforced concrete structures. Baseline values of soil corrosion parameters for Existing Fill and Native Soils are presented in Table 6.4-1.

Comparison with Caltrans criteria presented in Section 6.6.8 indicates that test results in S0033AR and S0071R exceed the criteria for chlorides, and four out of five tests are below the minimum resistivity. Comparison with durability requirements of the American Concrete Institute (ACI 318-11) indicates that baselined values are at least moderately corrosive to reinforced concrete structures.

The GI completed by the design-build Contractor for final design should identify specific corrosion protection requirements for each structure with further exploration and testing.

For bidding purposes, assume that 50% of all ground conditions are corrosive to concrete structures in accordance with the baseline values provided in Table 6.4-1. For costing, allocate this percentage evenly across foundations of all concrete structures. Assume that the remaining 50% of ground conditions are non-corrosive to buried concrete structures.

If designs incorporating steel exposed to soil are proposed, corrosion design requirements shall be determined using the baseline values in Table 6.4-1 and Table 6.4-2.

Table 6.4-1
Baseline Corrosion Parameters

Test	Test Reference	No. of Tests[a]	Range of Values	Mean Value	Standard Deviation	Baseline Value
Minimum Resistivity (ohm-cm)	ASTM G 57	5	399–6284	1830	2498	**500**
pH	ASTM D 4327	5	6.4–9.92	8	1.62	**6.5**
Sulfate (ppm)	ASTM D 4327	5	50–437	204	173	**450**
Chloride (ppm)	ASTM D4327	5	24–963	437	474	**950**

[a] All tests were conducted on bulk samples comprising soil from within the first 5 feet of depth; two samples from within Fresno County (S0030R and S0033AR) and three samples from within Tulare County (S0070R, S0071R, and S0073R).

6.4.2 Groundwater Chemistry

Groundwater corrosivity parameters are based on the results of four samples collected from the piezometers set during the PE4P field investigation. The mean value of parameters tested represents baseline conditions (Table 6.4-2). Refer to Section 6.4.1 for baseline of corrosion of in situ structures.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Table 6.4-2
Baseline Groundwater Chemistry Parameters

Test	Test Reference	Borehole ID				Baseline Value
		S0020R	S0068R	S0071R	S0072R	
pH	SM 4500-H^+B	7.7	6.8	11.9	9.5	**6.8**
Calcium (mg/L)	EPA 200.7	57.9	38.5	50.6	15.6	**40**
Bicarbonate Alkalinity as $CaCO_3$ (mg/L)	SM 2320B	325	158	749	45	**150**
Specific Conductance (umhos/cm)	SM 2510B	1050	570	4010	2080	**2000**
Total Dissolved Solids (mg/L)	SM 2320B	657	387	1580	1240	**1000**
Chloride (mg/L)	EPA 300.0	74.6	16.9	259	431	**200**
Sulfate as SO_4 (mg/L)	EPA 300.0	66.2	109	184	244	**150**

6.5 Engineering Parameters of the Subsurface Materials

6.5.1 Existing Fill and Near-Surface Soils

The primary purpose for providing baseline parameters for Existing Fill and Near Surface Soil is to facilitate the design of temporary and permanent foundations, pavements, and earthworks.

Few undisturbed laboratory tests were performed on Existing Fill because the bulk samples collected were highly disturbed and were taken from drilling cuttings. Laboratory tests performed included Modified Proctor Compaction, California Bearing Ratio, moisture content, and fines content.

Existing Fill was encountered in only 12 of 19 PE4P boreholes (Section 6.1.2). In areas without appreciable Existing Fill, such as long stretches of proposed embankment or at-grade (or near-grade) railway, surface works are likely to require disturbance or reworking of near-surface Native Soils. As a result, laboratory testing undertaken to inform pavement and earthworks design often comprises samples of both Existing Fill and shallow Native Soils. Therefore, baseline parameters presented in this section shall be applied to Existing Fill to the full depth over which they were encountered and near surface native soils up to 5 feet bgs.

In situ properties of Existing Fill and near-surface Native Soils , including total unit weight and natural water content, are presented in Error! Reference source not found. and are based on too-few tests to develop statistically significant conclusions. Despite the limited number of tests, the total unit weight data provide mean values generally within expectation for the soils types encountered, and these mean values have been adopted as the baseline value.

Moisture content will vary significantly by season and recency of rainfall. The as-measured, in situ moisture contents for coarse-grained materials approximate the optimum moisture content from compaction tests Table 6.5-2. The approximation is coincidental and should not lead the bidder to assume that no moisture conditioning of site soils will be required. Moisture conditioning assumptions are described further in Section 6.6.1.

Table 6.5-1
Baseline In Situ Properties for Existing Fill and Near-Surface Native Soils

		Total Unit Weight[a] (γ_t)	**Natural Water Content (w_c)**
Fresno County			
	No. of Tests	3	9
	Range	111.8–129 pcf	2.2–17.5%
	Assumed Baseline	120 pcf	8.4%
Tulare County			
COARSE	No. of Tests	2	3
COARSE	Range	110.4–124.1 pcf	6.4–11.9%
COARSE	Assumed Baseline	117 pcf	9.1%
FINE	No. of Tests	4	8
FINE	Range	119.2–126.6 pcf	15.5–34.0%
FINE	Assumed Baseline	123 pcf	24%

[a] Based on modified California sampler data

Baseline compaction parameters of Existing Fill and near-surface Native Soils are provided in Table 6.5-2. The number of tests (e.g., 17 compaction tests from all of Fresno and Tulare County combined) is limited, and the baseline values presented are typically mean values of the respective data sets.

Baseline in situ strength parameters for Existing Fill and near-surface Native Soils are provided in Table 6.5-3. No laboratory strength testing was undertaken on near surface materials, and the baseline strength parameters provided rely on engineering judgment for similar materials based on perceived composition and in situ density/consistency.

Bulking/swell factors used to estimate earthwork volumes typically range between 10% for sand and gravel to about 30% for clay. Shrinkage factors range from about 10% for sand to about 30% for clay. For bidding purposes, assume Existing Fill has a bulking/swell factor of 20% and a shrinkage factor of 10%.

Table 6.5-2
Baseline Earthworks Parameters for Existing Fill and Near Surface Soils

		Fines Content	Maximum Dry Density ($\gamma_{d,max}$)	Optimum Moisture Content (w_o)	California Bearing Ratio	R-Value
Fresno County						
	No. of Tests	7	7	7	4	6
	Range	12–49%	119.1–124.5 pcf	7.4–10.7%	15–48%	61–73
	Assumed Baseline	**20%**	**121 pcf**	**8.6%**	**10%** [a]	**30** [a]
Tulare County						
COARSE	**No. of Tests**	2	**5**	**5**	**-**[b]	5
	Range	39–42.2%	113.4–128.4 pcf	9.1–14.4%	**-**[b]	9–60
	Assumed Baseline	**30%**	**121 pcf**	**11.5%**	**5%**[b]	**10**
FINE	**No. of Tests**	**–**	**5**	**5**	**2**	4
	Range	**–**	117.2–128.9 pcf	8.5–13.1%	2.3–2.5%	4–22
	Assumed Baseline	**–**	**123 pcf**	**10.4%**	**2.0%**	**5**

[a] Baseline CBR and R-Value not to exceed maximum values for native soils as permitted by local jurisdictions. Test results shown may be influenced by roadway subgrade material in samples. Local jurisdictional maximum R-Values must govern for pavement section design, and superseded any baseline herein.

[b] Indicates remoulded laboratory tests were not performed; values presented are based on engineering judgment. The selective grading for reuse of coarse and fine layers during earthworks will be highly dependent upon layer thickness and construction means and methods.

Notes:

Baseline values are less than mean values and have been chosen to be more representative of likely soil conditions.

Soils tested comprise hand auger samples collected over depths of 0ft to 10ft, and borings were typically conducted on roadway shoulders. Conditions elsewhere (e.g., nearby agricultural land) may vary.

Coarse grained soil results in Tulare County likely reflect the influence of high fines contents.

Table 6.5-3
Baseline In Situ Strength Parameters for Existing Fill and Near Surface Soils

Location	Soil Type	Effective Strength Parameters [a]		Undrained Shear Strength [a], s_u (psf)
		Friction Angle, Φ', (°)	**Cohesion (psf)**	
Fresno County	Coarse	30	- [b]	N/A
Tulare County	Coarse	29	- [b]	N/A
	Fine	28	- [b]	1,000

[a] Laboratory testing to assess strength of existing fill and near surface soil was not undertaken. Values presented are based on engineering judgment for typical values based on material type and perceived in situ density or consistency. Strength of reworked material will vary by compactive effort and moisture conditions.

[b] Effective cohesion shall be taken are zero, except for purposes on earthworks slope stability assessment, where a value of 50psf shall be adopted.

6.5.2 Native Soils of Fresno County

The baseline description of Native Soil encountered in Fresno County provided in Section 6.1.3 indicated predominantly granular soils with increasing fine content below approximately 25 feet bgs. A single set of engineering parameters for the native soils, above and below a depth of 25 feet, is provided as a baseline for design across the entire county. While it is likely that conditions will deviate along the alignment, a review of the available data suggests that the present information is generally insufficient to justify specific distinctions for individual structures or stretches of alignment.

The presence of fine grained soils in Fresno County is indicated by the soil type distributions shown in Figure 6.1-5 and Figure 6.1-6. The nature, frequency and distribution of the fine-grained soils encountered during the GI suggests that the design impact of ignoring these fines will, for most applications, be minor. No baselined engineering parameters have been provided for fine grained soils in Fresno County.

Baseline parameters for native soils of Fresno County are presented in Table 6.5-4. The range of conditions and uncertainties for these parameters are described in Appendix A.

Table 6.5-4
Baseline Engineering Properties for Native Soils in Fresno County

Applicable Depth Regime	Value	Total Unit Weight γ_t (pcf)	Soil Modulus E_s [a] (tsf)	Corrected Blow Count SPT N_{60} (bpf)	CPT Tip Resistance q_c (tsf)	Effective Friction Angle Φ' [b] (deg)	Effective Cohesion Intercept c' [b, c] (psf)	Shear Wave Velocity V_s (ft/sec)
Fresno County Above 25 ft	Range	101–129	62–>2,000	10–R	0.2–1,183	30–51	140–1,380	523–2,488
	Baseline	**110**	**300**	**18**	**100**	**32**	**80**	**600**
Fresno County Below 25 ft	Range	102–135	42–>2,000	23–R	0.02–1,111	10–43	100–3,640	588–2,252
	Baseline	**120**	**500**	**40**	**200**	**36**	**60**	**1,000**

[a] Range of Soil Modulus reported derives from correlation with SPT $N_{1(60)}$ and CPT qc data. Baseline values rely on CPT data and typical values based on relative density as per AASHTO.

[b] Ranges given are based on laboratory data only, primarily direct shear tests

[c] Effective Cohesion is an apparent cohesion used only to adjust for the non-linearity at low stresses typical of simplified Mohr-Coulomb failure criteria. For baseline purposes, effective cohesion shall be ignored for all failure surfaces through or along undisturbed native soils confined by less than 10 feet of overburden.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

6.5.2.1 Standard Penetration Test Blow Count

The baseline SPT N_{60} blow count shown in Table 6.5-4 is selected from the SPT data set for the entire county as the median above 25 feet, and the mean minus one standard deviation below 25 feet.

SPT blow counts were recorded during soil sampling in boreholes and corrected to SPT N_{60} values using the results of hammer efficiency measurements recorded during the site exploration. For comparison, CPT tip resistance data was correlated to equivalent SPT N_{60} values as described in Appendix A.

Histograms and statistical data of SPT N_{60} are presented in Appendix A. Histogram plots were capped at a maximum value of 100 blows per foot.

Hardpan soils are typically encountered within 40 feet bgs and can be identified from N_{60} values corrected for overburden (referred to as $(N_1)_{60}$). Figure 6.5-1 shows the variation of $(N_1)_{60}$ with depth for all SPT data; high resistance values typically above 60 can be indicative of hardpan soil. Hardpan soils were encountered during the PE4P GI in several boreholes as indicated.

Shallow refusal was also experienced in several CPTs— S0045ACPT, S0046CPT (at 20 feet), S0094ACPT (at 20 feet), and S0098CPT (30 feet)–requiring predrilling to prevent damage to the equipment. The results of boreholes drilled adjacent to S0045ACPT, and S0098CPT indicates the Hardpan material is expected to consist of SM, CL-ML, and SP with N_{60} values greater than 60 and moderate to strong cementation. These layers varied from about 5 to 8 feet thick.

As a baseline, assume that 5 feet of hardpan is present at the depths indicated by the boreholes and CPTs cited above, in two primary areas: 1) between Clovis Avenue and SR43, 2) between Clayton Avenue and Lincoln Avenue,. Further assume that the extent of hardpan spans over the full width of the proposed works in these areas.

Resistance at deeper levels, likely associated with varying degrees of cementation, was experienced in the majority of boreholes at depths ranging from 50 feet to over 130 feet, and more concentrated between 70 feet and 110 feet. Conditions and design considerations will vary by location, and no baseline is provided.

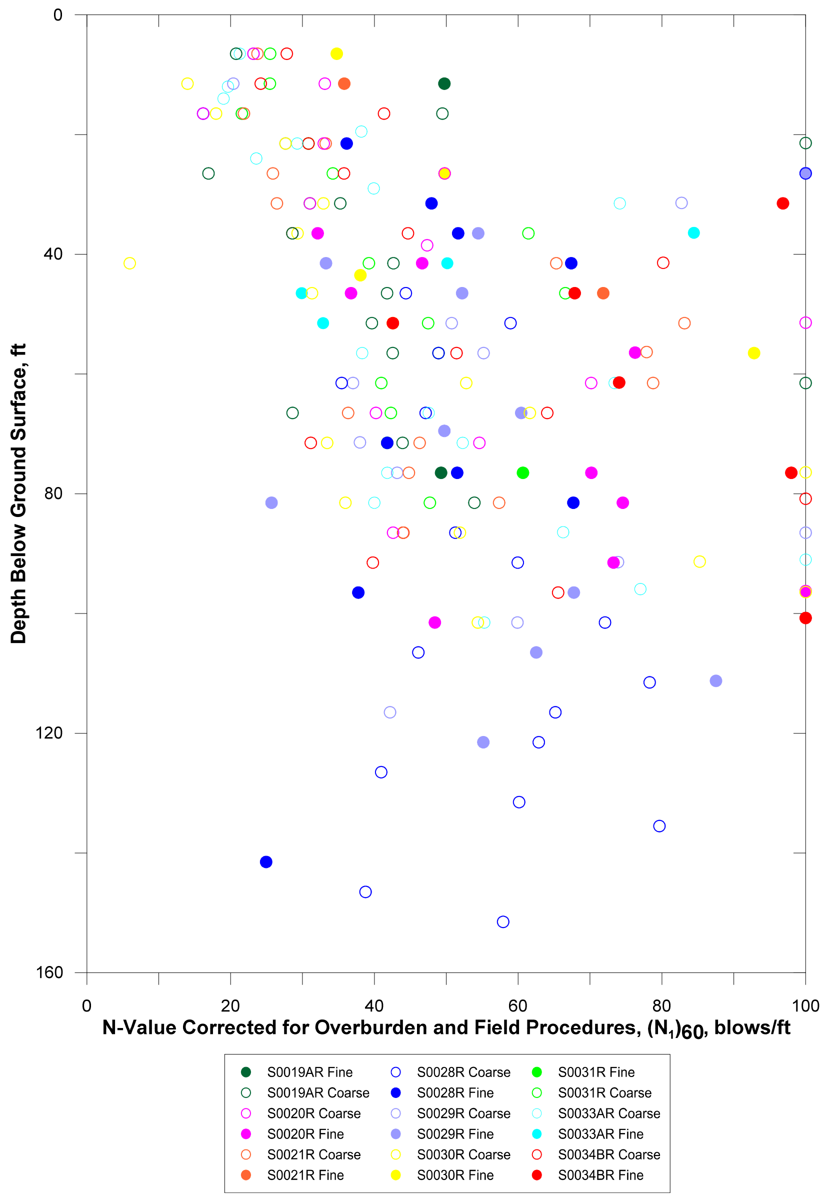

Figure 6.5-1
SPT $(N_1)_{60}$ Results for Fresno County

6.5.2.2 Cone Penetration Test Tip Resistance

The baseline CPT tip resistance (q_c) in Table 6.5-4 is selected as the median q_c value from the CPT data below 25 feet.

The value above 25 feet was reduced from the mean to be more consistent with $(q_c/p_a)/N_{60}$ = 5.0 for the predominantly sandy soil that is anticipated.

CPT tip resistance data for native soils above and below 25 feet, including mean, median, and standard deviation results, are presented in Appendix A.

6.5.2.3 Unit Weight

The total unit weight baseline values shown in Table 6.5-4 were selected as the mean value from 17 and 45 drilling samples taken above and below a depth of 25 feet, respectively.

Histograms of CPT correlated unit weight and densities from drilling samples are presented in Appendix A.

6.5.2.4 Effective Shear Strength

Shear strength parameters for Fresno County include effective friction angle (Φ’) and effective cohesion (c’). The effective friction angle for the predominantly coarse-grained soil of Fresno County was determined from CPT and SPT blowcount correlations, as well as from the results of triaxial consolidated drained tests (TXCD) and direct shear (DS) tests on driven samples from California Modified and piston samplers. The statistical results of the CPT and SPT correlations and laboratory test data are presented in Appendix A.

Strength parameters were estimated from the results of 26 direct shear tests on soil samples collected during the field exploration. The direct shear test results of tests with excessively high effective cohesions (> 1000 psf) were not used in developing the strength baseline and are assumed to misrepresent in situ conditions.

For reference purposes, the baseline effective friction angle and effective cohesion indicated in Table 6.5-4 are illustrated graphically in Figure 6.5-2, with the direct shear tests results.

Generally lower bound values to the laboratory data have been selected for baseline values, and further reduced by engineering judgment to values closer to geotechnical expectation and practice. This is considered appropriate for bidding purposes, due to the high sensitivity of many designs to these parameters, the potential for strength to vary along the alignment, and the relatively limited amount of testing.

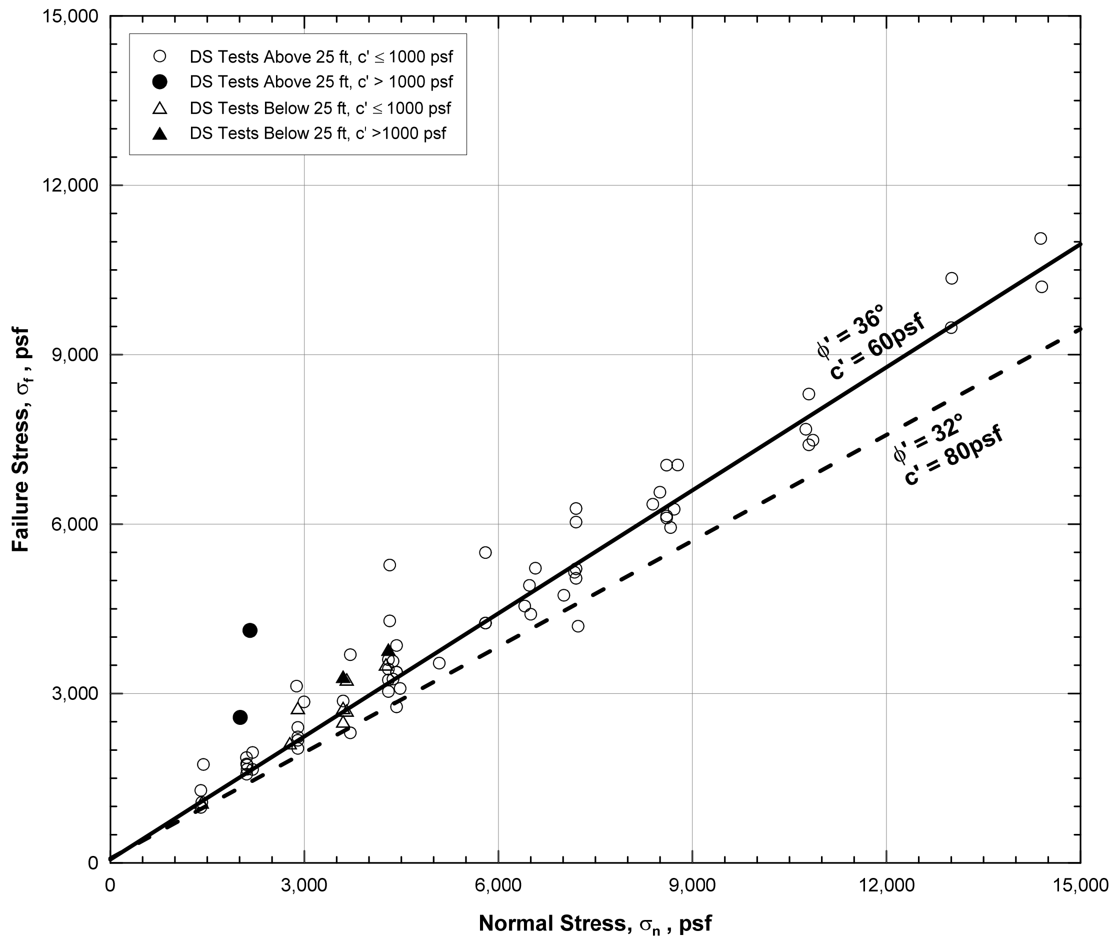

Figure 6.5-2

Results of Direct Shear Tests for Native Soil Samples Obtained in Fresno County

1.4.1.1. Soil Modulus

Soil modulus (E_s) was estimated using correlations with $N_{1(60)}$ and CPT q_c, as detailed in Appendix A. The baselined values in Table 6.5-4 approximate typical soil moduli for the in situ densities inferred from GI results. The baseline represents a lower bound to the CPT correlation and a mid to upper bound of the SPT correlation. In general, the SPT correlation resulted in lower estimates of soil modulus.

Typical values for sand as presented by AASHTO (2010) are presented in Table 6.5-5. The baselined values above and below 25 feet depth are representative of loose to medium-dense sand and medium-dense to dense sand, respectively.

Table 6.5-5

Published Soil Modulus (AASHTO 2010)

Soil Modulus, E_s (tsf)
Silt
20 to 200

Table 6.5-5
Published Soil Modulus (AASHTO 2010)

Soil Modulus, E_s (tsf)	
Sand	
Loose	100 to 300
Medium Dense	300 to 500
Dense	500 to 800

The results of CPT-based soil modulus estimates are presented graphically in Figure 6.5-3. Histograms and other statistical data used to determine soil modulus from SPT and CPT correlations are presented in Appendix A.

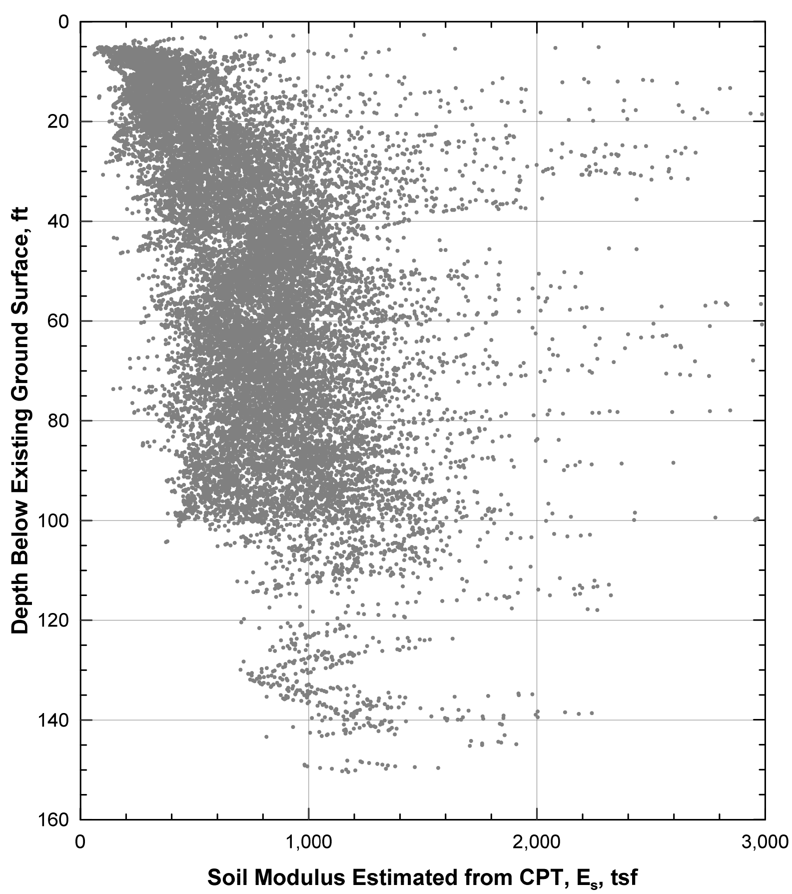

Figure 6.5-3
Soil Modulus Correlations from CPT Data of Fresno County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

6.5.2.5 Shear Wave Velocity

Shear wave velocities averaged over the upper 100 feet (~30 meters) of soil, V_{s30}, are presented in the GDR. Baseline V_s values based on the available data are shown on Table 6.5-4. For soil above and below 25 feet in Fresno County, baseline V_s of 600 and 1000 feet/sec were selected.

Figure 6.5-4 shows the baseline values overlain on the V_s profiles measured during the PE4P GI. In Fresno County, measurements were taken in S0053CPT, S0088CPT, and S0102CPT, and via PS logging in S0028R. The seismic Site Class boundary between Class C and Class D soil is shown for reference only.

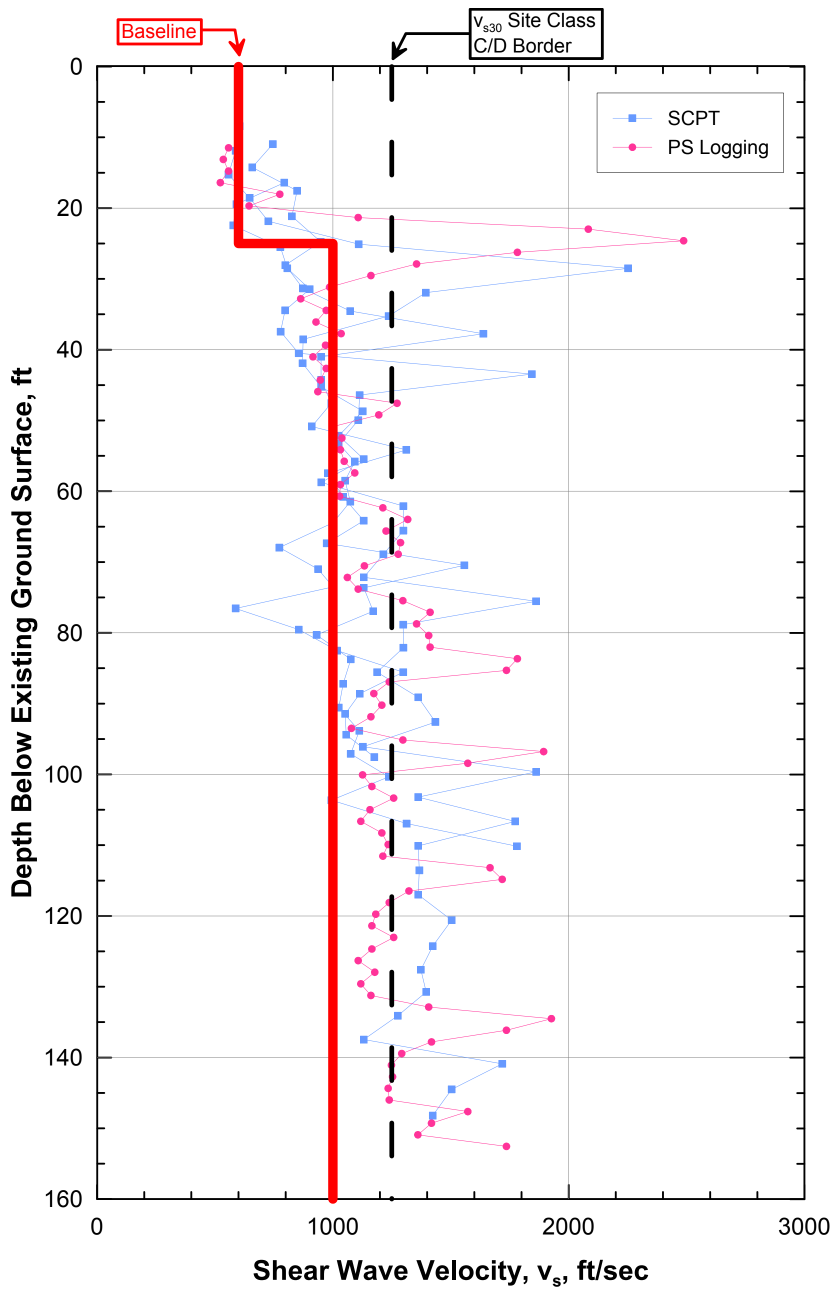

Figure 6.5-4
Shear Wave Velocity Measurements and Baseline Value in Fresno County

6.5.2.6 Modulus of Vertical Subgrade Reaction

Figure 6.5-5 shows the range of Modulus of Vertical Subgrade Reaction (k'_v) applicable for sands. The baseline subgrade modulus is determined from the baseline SPT N_{60} blow count correlated to the typical vertical subgrade reaction modulus values shown in Figure 6.5-5. A bi-linear relationship between subgrade modulus and relative density was utilized.

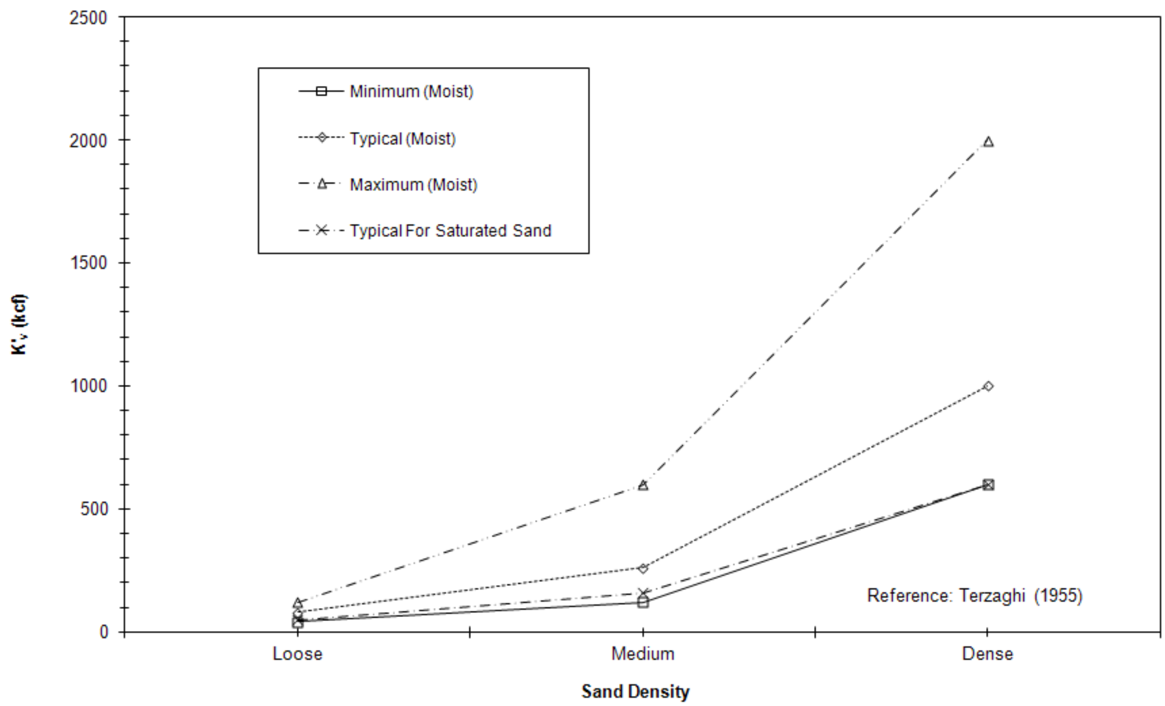

Figure 6.5-5
Modulus of Vertical Subgrade Reaction

6.5.2.7 L-Pile Parameter: Modulus of Horizontal Subgrade Reaction

Typical values of Modulus of Horizontal Subgrade Reaction (k_h) for granular soil range from 20 to 225 pounds per cubic inch based on an assessment of the relative density of the sand and the effect of a submerged or dry condition (FHWA-NHI-10-16). Typical values of k_h published by the American Petroleum Institute (API 1987) are shown on Table 6.5-6.

Table 6.5-6
Static Modulus of Horizontal Subgrade Reaction, k_h (API 1987)

	Subgrade Reaction k_h by Relative Density (pci)		
	Loose	**Medium Dense**	**Dense**
Sand Below Water Table	20	60	125
Sand Above Water Table	25	90	225

For bidding purposes, assume a modulus of horizontal subgrade reaction (k_h) varies above and below 25 feet bgs. For soils above 25 feet bgs, a baseline value of 40 pci for static conditions and 20 pci for cyclic loading may be adopted. For soils below 25 feet bgs, a baseline value of 80 pci for static conditions and 40 pci for cyclic conditions may be adopted. These values do not assume liquefied soil conditions. Usage of these parameters is intended for and limited only to lateral pile analysis using L-Pile software.

6.5.3 Native Soils of Tulare County

The baseline description of Native Soil in Tulare County provided in Section 6.1.3 indicated interbedded coarse- and fine-grained soils, and provided baselines on the proportions and distribution of these primary soil categories along the alignment.

A single set of engineering parameters for the native soils, above and below a depth of 35 feet, is provided as a baseline for design across the entire county. While it is likely that conditions will deviate along the alignment, a review of the available data suggests that the present information is generally insufficient to justify further distinctions for individual structures.

Baseline parameters for native soils of Tulare County are presented in Table 6.5-7**.** The range of conditions and uncertainties for these parameters are described in Appendix A.

Table 6.5-7

Baseline Engineering Properties for Native Soil in Tulare County

Material	Applicable Depth Regime	Value	Total Unit Weight γ_t (pcf)	Soil Modulus E_s (tsf)	Corrected Blow Count SPT N_{60} (bpf)	CPT Tip Resistance q_c (tsf)	Effective Friction Angle Φ' (deg)	Effective Cohesion Intercept[2] c' (psf)	Undrained Shear Strength[3] s_u (psf)	Shear Wave Velocity V_s (ft/sec)
Coarse-grained soils	Above 35 ft	Range	100–136	4–936	12–64	5–468	20–39	100–1000	–	392–1232
		Baseline	**120**	**300**	**25**	**100**	**32**	**100**	**–**	**600**
	Below 35 ft	Range	122–136	15–1205	10–99	8–602	22–40	140–1000	–	680–1508
		Baseline	**128**	**500**	**50**	**200**	**36**	**150**	**–**	**1000**
Fine-grained soils [4]	Above 35 ft	Range	119–136	Varies [1]	7–99	4–104	31–40	250–1000	1880–3262	392–1232
		Baseline	**125**	**300**	**15**	**25**	**30**	**100**	**2,400**	**600**
	Below 35 ft	Range	99–135	Varies [1]	8–99	7–365	27–30	780–1000	1065–5261	680–1508
		Baseline	**125**	**500**	**30**	**50**	**32**	**250**	**3,200**	**1,000**

[1] Soil modulus for fine-grained soil assumes 300 * s_u. For coarse-grained soil, Modulus reported relies on correlation with CPT data and typical values based on relative density as per AASHTO.

[2] For coarse-grained soil, cohesion is either representative of cementation, or is an apparent cohesion used only to adjust for the non-linearity at low stresses typical of simplified Mohr-Coulomb failure criteria. Cementation may be subject to softening when exposed to elevated groundwater or perched water. For baseline purposes, effective cohesion shall be ignored for all failure surfaces through or along undisturbed native soils confined by less than 10ft of overburden.

[3] Ranges given are based on laboratory TXUU data, baseline further considers $c_u = (q_c - \sigma_v) / N_k$, where $N_k = 17$.

[4] Baseline of consolidation parameters for fine grained native soil in Tulare County are provided in Section 6.5.3.10.

6.5.3.1 Standard Penetration Test Blow Count

The baseline SPT N_{60} blow count shown in Table 6.5-7 is selected as the median from the SPT data set for fine-grained native soil above and below 35 feet. For coarse-grained native soil above and below 35 feet, the baseline values are 5 and 10 units below the mean value, respectively, to account for the wider spread of data.

SPT blow counts were recorded during soil sampling in boreholes and corrected to SPT N_{60} values using the results of hammer efficiency measurements recorded during the site exploration. For comparison, CPT tip resistance data was correlated to equivalent SPT N_{60} values as described in Appendix A.

Histograms and statistical data of SPT N_{60} for Tulare County are shown in Table 6.5-5 are presented in Appendix A. Histogram plots were capped at a maximum value of 99 blows per foot.

Hardpan soils are typically located within 40 feet bgs, and can be identified from N_{60} values corrected for overburden, referred to as $(N_1)_{60}$. Figure 6.5-6 shows the variation of $(N_1)_{60}$ with depth for all SPT data; high resistance values typically above 60 can be indicative of hardpan soil. The data indicates potential hardpan at 35 feet in S0066R and at 30 feet in S0068R. Furthermore, none of the CPTs in Tulare County required predrilling due to shallow cone tip refusal. It is therefore concluded that hardpan is less prevalent in Tulare than Fresno County.

As a baseline, it can be assumed that hardpan exists over less than 2% of the alignment area in Tulare County.

Resistance at deeper levels, likely associated with varying degrees of cementation, was experienced in the majority of boreholes at depths ranging from 50 feet to 140 feet. Conditions and design considerations will vary by location, and no baseline is provided.

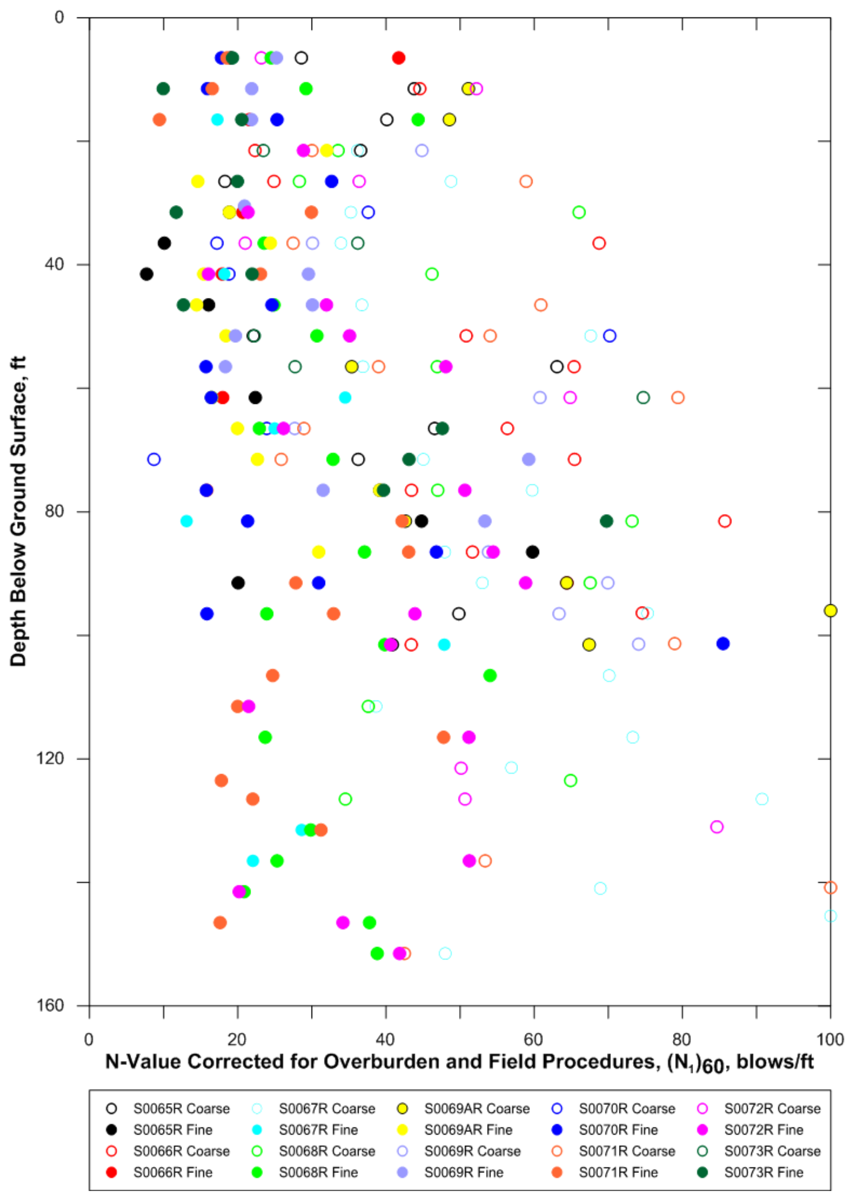

Figure 6.5-6
SPT $(N_1)_{60}$ Results for Tulare County

6.5.3.2 Cone Penetration Test Tip Resistance

The baseline CPT tip resistance (q_c) in Table 6.5-7 is selected as the mean q_c value from the CPT data set for fine-grained Native Soil in Tulare County. For coarse-grained Native Soil of Tulare County, the baseline q_c value is selected as the average of the mean and median values above 35 feet, and as the mean minus (slightly less than) one half standard deviation below 35 feet, to account to the spread in the data.

CPT tip resistance data for each structure, including mean, median, and standard deviation results, are presented in Appendix A.

6.5.3.3 Unit Weight

The total unit weights baseline shown in Table 6.5-7 were selected as the mean values from borehole sampler data gathered during the investigation.

Histograms of CPT correlated unit weight and densities from drilling samples are presented in Appendix A.

6.5.3.4 Effective Shear Strength

Effective shear strength parameters include effective friction angle (Φ′) and effective cohesion (c′). The effective friction angle for predominantly coarse-grained native soil in Tulare County (refer to Table 6.1-4) was determined from CPT and SPT blow count correlations, as well as from the results of 16 direct shear tests on physical samples. The statistical results of the CPT and SPT correlations and laboratory test data are presented in Appendix A.

Effective friction angle and effective cohesion for fine gained native sol in Tulare County was determined from the results of 6 direct shear tests, and correlation with SPT data, also presented in Appendix A.

Generally lower bound values to the laboratory data have been selected for baseline values, and further reduced by engineering judgment to values closer to geotechnical expectation and practice. This is considered appropriate for bidding purposes, due to the high sensitivity of many designs to these parameters, the potential for strength to vary along the alignment, and the relatively limited amount of testing.

The baseline effective strength parameters, comprising effective friction angle and effective cohesion, are presented in Table 6.5-7.

A number of tests results exhibited high effective cohesions, possibly a consequence of testing method or of cementation in the coarse-grained native soils. Cemented behavior in cohesionless soils can result in brittle failure, and can soften if subject to wetting such as from perched or rising groundwater, or from cyclic loading over time.

Figure 6.5-7 shows the stress envelopes from all direct shear tests performed, including the baseline effective strength envelopes from Table 6.5-7, for reference purposes.

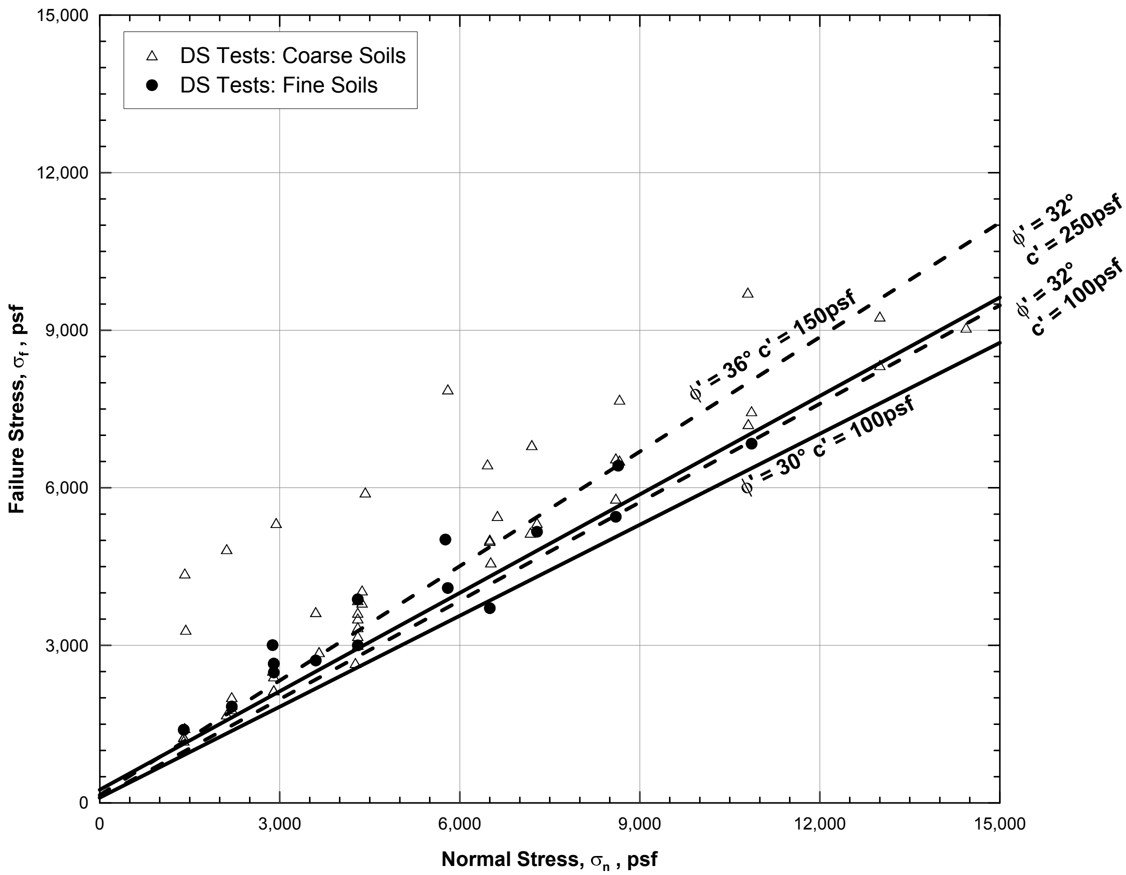

Figure 6.5-7

Results of Direct Shear Tests for Native Soils Sampled in Tulare County

6.5.3.5 Undrained Shear Strength

Undrained shear strength is a design parameter relevant to fine-grained materials. Fine-grained materials can exhibit cohesive behavior that retards the drainage of pore water in saturated or partially-saturated soil. This results in an 'undrained' response to applied load, whereby excess pore water pressure is generated, and the initial resistance provided by the soil is represented by the undrained shear strength.

The usage of drained or undrained strength parameters is application-specific and to be determined by the design builder.

Undrained shear strength for fine-grained native soil of Tulare County was determined from triaxial unconsolidated undrained (TXUU) shear strength tests on 9 borehole samples taken above 35 feet bgs, and 26 tests on samples from below 35 feet bgs. Undrained shear strength was also estimated by correlation with CPT q_c data. The details and statistics for this data are presented in Appendix A.

The CPT data are indicative of higher undrained shear strength than suggested by the TXUU tests results, particularly below 35 feet. This is not uncommon, as laboratory samples are more prone to disturbance and relaxation during transport, extrusion, and testing. By comparison, penetration response during CPT testing can be more representative of undisturbed in situ conditions. Undrained shear strength for fine grained Native Soil in Tulare County, as estimated from CPT data, is presented in Figure 6.5-8.

The baseline undrained shear strength for fine-grained native soil in Tulare County is presented in Table 6.5-7.

The baseline value above 35 feet is slightly less than the mean value from the laboratory testing, 20% below the mean value of the CPT correlation, and is representative of a stiff material consistent with observations of the investigation. The baseline value below 35 feet is approximately 20% above the mean value of the laboratory tests, 20% below the mean value of the CPT correlation, and representative of a very stiff material.

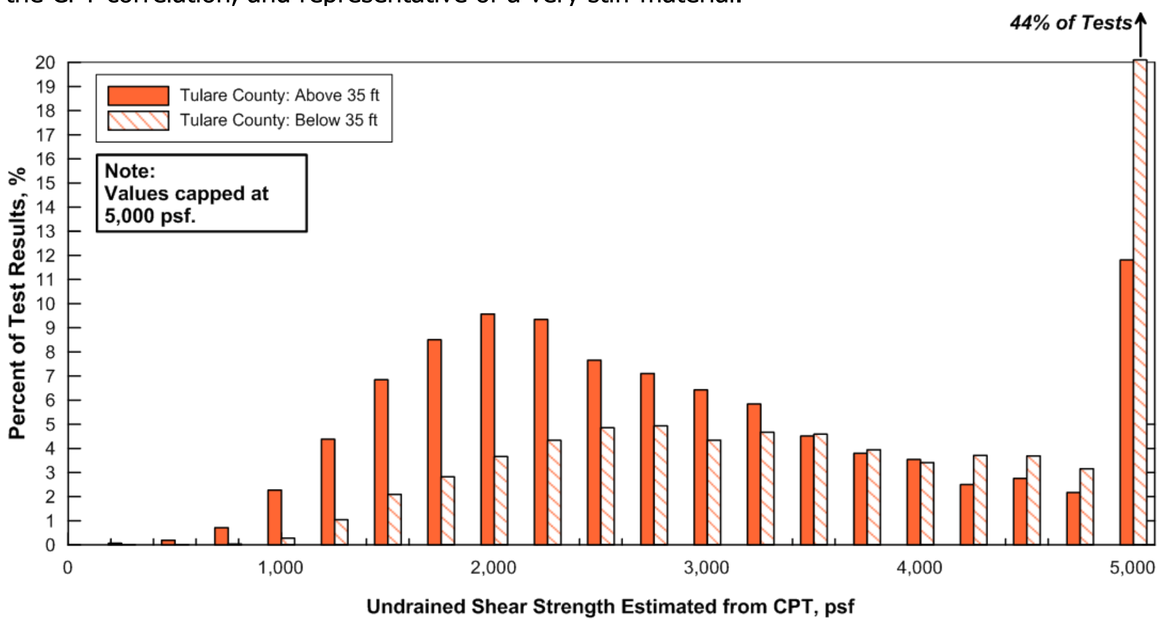

Figure 6.5-8

Undrained Shear Strength of Fine-Grained Soil of Tulare County Correlated from CPT_{qc}

6.5.3.6 Soil Modulus

Soil Modulus (E_s) was estimated using $(N_1)_{60}$ and CPT based correlations as shown in Appendix A.

Baseline values for coarse- and fine-grained native soil above and below 35 feet bgs in Tulare County are presented in Table 6.5-7.

For coarse-grained native soil, the CPT correlation yield greater estimates of soil modulus values than the SPT correlation. Baselined values of soil modulus reflect a lower bound to the CPT estimated and an upper bound to the SPT estimates, and are representative of a loose to medium-dense sand above 35ft and a medium-dense to dense sand below 35ft in accordance with published ranges (Table 6.5-5) and consistent with observations during the GI.

For the fine-grained native soil, soil modulus was assessed by applying the correlation $E_s = 300 * s_u$ (Lunne, 1997) to undrained shear strength (s_u) from both laboratory test results and correlation with CPT q_c. The baseline soil modulus closely approximates 300 times the baseline undrained shear strength for above and below 35ft depths, representative of stiff and very stiff clays, respectively. The SPT and CPT correlations indicate Soil Modulus values lower than would be expected for stiff to very stiff soils. It is expected that overconsolidated fine grained materials will behave nearly elastically when loaded, and will be represented well by the correlation $E_s = 300 * s_u$ (Lunne et al, 1997). Applying this correlation to the baseline values of undrained shear strength from Table 6.5-7 yields a soil modulus of 360tsf above 35 feet, and 480tsf below 35 feet.

Histograms and other statistical data used to determine Soil Modulus from SPT and CPT correlations are presented in Appendix A. However these data have been largely discounted from the selection of the baseline value in favor of correlations with undrained shear strength (Lunne et al, 1997).

6.5.3.7 Shear Wave Velocity

Shear wave velocities averaged over the upper 100 feet of soil, V_{s30}, are presented in the GDR. Baseline shear wave velocities based on the available data are shown on Table 6.5-5. For soil above and below 35 feet in Tulare County, baseline values of 600 and 1000 feet/sec were selected.

Figure 6.5-9 shows the baseline values overlain on the shear wave velocity profiles measured during the PE4P GI. In Tulare County, measurements were taken in S0194CPT, S0216CPT, and S0226CPT, and via PS logging in S0067R and S0072R. The seismic Site Class boundary between Class C and Class D soil is shown for reference only.

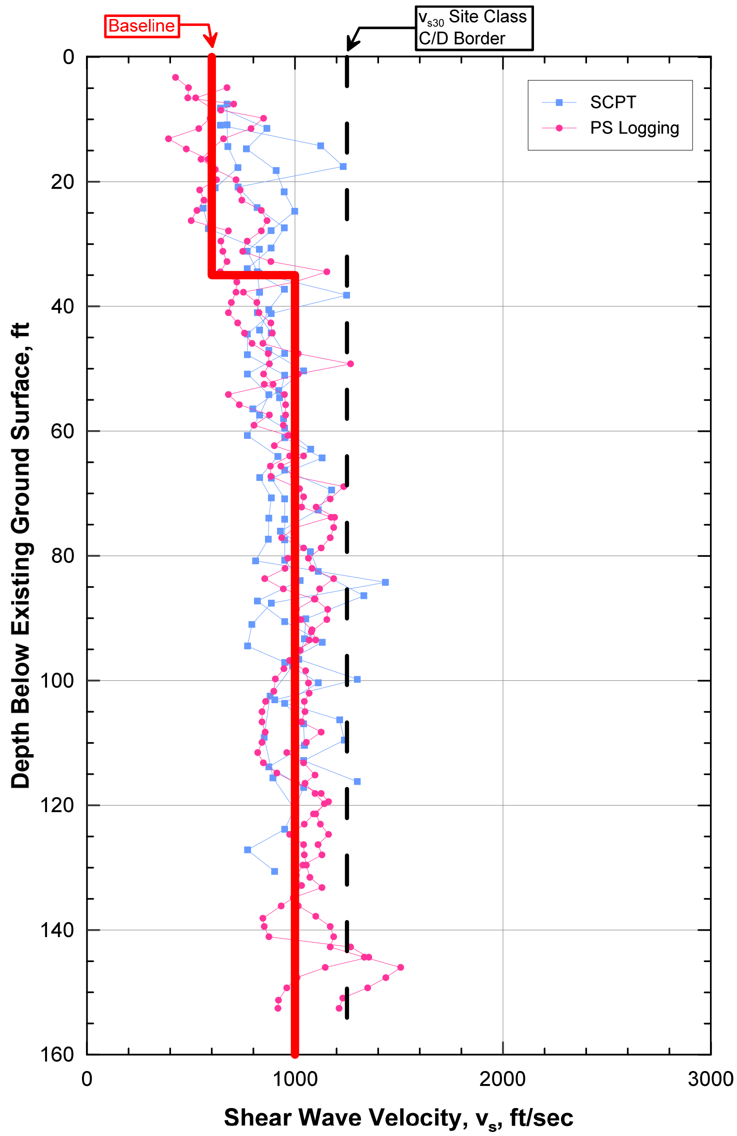

Figure 6.5-9
Shear Wave Velocity Measurements and Baseline Value in Tulare County

6.5.3.8 Modulus of Vertical Subgrade Reaction

Figure 6.5-10 shows the range of Modulus of Vertical Subgrade Reaction (K'_v) applicable for sands. The baseline subgrade is determined from the baseline SPT N_{60} blow count correlated to the typical vertical subgrade reaction modulus values shown in Figure 6.5-10. A bi-linear relationship between subgrade modulus and relative density was utilized.

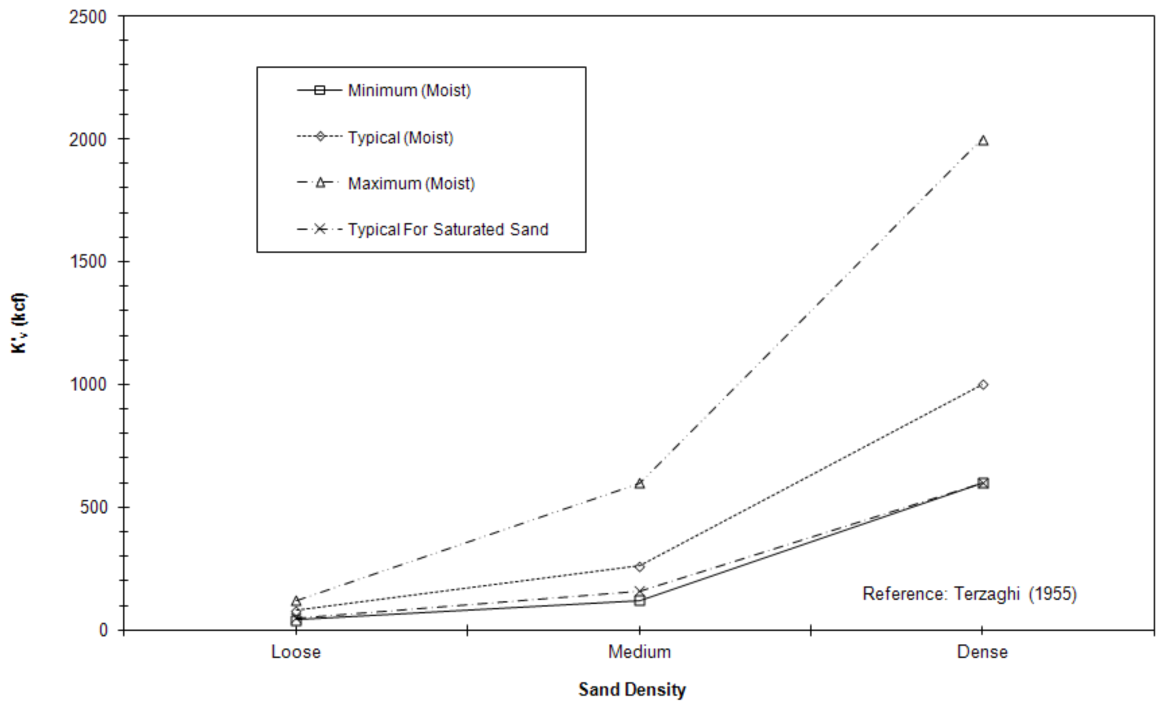

Figure 6.5-10
Modulus of Vertical Subgrade Reaction

6.5.3.9 L-Pile Parameter: Modulus of Horizontal Subgrade Reaction

Typical values of Modulus of Horizontal Subgrade Reaction (k_h) for granular soil range from 20 to 225 pounds per cubic inch based on an assessment of the relative density of the sand and the effect of a submerged or dry condition (FHWA-NHI-10-16). Typical values of kh published by the American Petroleum Institute (API 1987) are as presented previously in shown on Table 6.5-6.

For bidding purposes, assume a modulus of horizontal subgrade reaction (k_h) for coarse-grained native soil of Tulare County identical to those values proposed for Fresno County above 25 feet. Similarly, for coarse-grained soil of Tulare County below 35 feet adopt the same value as was baseline for Fresno County below 25 feet. These values do not assume liquefied soil conditions.

For fine-grained native soil of Tulare County, assume a k-value of 1,500 psf above 35 feet bgs, and 2,500 psf below 35 feet bgs.

Usage of these parameters is intended for and limited only to lateral pile analysis using L-Pile software.

6.5.3.10 Consolidation Parameters

Consolidation testing was performed on six samples of fine-grained, native soil from Tulare County. Tested samples ranged in depth from 12 feet to 123 feet and materials ML, CL, and CH.

Baseline consolidation parameters are provided in Table 6.5-8.

Refer to Section 6.6.9 for further discussion and performance baseline related to long-term settlement.

Table 6.5-8
Baseline Consolidation Parameters for Fine-Grained Native Soil in Tulare County

	(Virgin) Compression Index, C_c	Unload/ Reload Compression Index, C_r	Over-Consolidation Ratio, OCR	Coefficient of Consoldiation, C_v (ft^2/day)
Range of lab data	0.13 – 0.32	0.008 – 0.025	1.7 – 14.3	*
Baseline	0.2	0.02	4.0 (0 to 50′) 3.0 (50 to 100′) 2.0 (>100′)	1. 0 (reload) 0.2 (virgin)

* varies based on interpretation
A baseline statement for long-term settlement from deep soil behavior is provided in Section 6.6.9.

6.6 Baseline Soil Behavior during Earthwork

The soil behavior during earthwork will be a function of the equipment and means and methods selected by the Contractor.

6.6.1 Existing Fill

For bidding purposes, assume Existing Fill is loose to medium dense and soft to stiff and can be excavated with conventional grading equipment such as dozers, scrapers, and track mounted excavators. Where excavated vertically, Existing Fill will not remain stable. Excavations in Existing Fill will be prone to raveling within a few minutes where it is dry, and will flow where it is wet. It is anticipated that sloped cuts or temporary shoring will be required to maintain stability of excavation in Existing Fill.

Existing Fill will require moisture conditioning prior to reuse and recompaction to achieve desired density. This will require adding water to soil that is dry of the optimum moisture content and air drying soil that is wet of the optimum moisture content. Air drying during periods of rain (November through March) is assumed to be impractical. Stabilization through addition of lime may be applicable in some areas of Tulare County, where fine grained soils are of sufficiently clayey.

In general, cement or lime treatment may be appropriate, but for bidding purposes assume it will not be necessary.

6.6.2 Hardpan

Hardpan may be encountered during in Fresno County, at the depths and locations presented in Section 6.5.2.1 and 6.5.3.1.

Where encountered, it is difficult to excavate with conventional equipment such a truck mounted excavators and scrapers. Hardpan can also be difficult to excavate with conventional solid flight auger.

Hardpan is difficult to excavate when dry but may lose strength and become easily remolded when saturated leaving to reduced bearing and lateral capacity. For this reason, hardpan within 5 feet of the ground surface, if encountered, should not be relied upon for support of permanent structures.

The hardpan encountered during investigation is likely deeper than required excavation, however it may impact productivity of drilled shafts and complicate installation of driven piles. If driven piles are considered by the design-build Contractor in the locations subject to this baseline, it should be assumed that predrilling with a smaller (than pile size) diameter auger will be necessary, and the frictional resistance above hardpan level shall be reduced by 50%.

6.6.3 Cementation (Rippability)

The predominant coarse-grained soil of Fresno County exhibits no cementation to weak cementation according to the *Soil and Rock Logging, Classification, and Presentation Manual* (Caltrans 2010) as shown in Table 6.6-1. In Tulare County, the coarse-grained material is similar in this regard to Fresno County, however the fine-grained materials exhibit no cementation to strong cementation.

For bidding purposes, assume that Existing Fill, Native Soils in Fresno County, and coarse-grained Native Soil in Tulare County exhibit weak cementation.

Table 6.6-1
Cementation Criteria (Caltrans 2010)

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

6.6.4 Stability

For bidding purposes, Native Soils above groundwater are assumed to be firm and to remain stable for sufficient time to allow for temporary shoring installation. Native Soils below the groundwater, or above the phreatic surface but subject to locally higher (perched) water, will experience sloughing or running conditions. Shallow areas may thus require benching or battering to provide stable conditions. Where deep foundations extend below the groundwater level for construction, temporary casing and/or drilling slurry will be required.

6.6.5 Shrink/Swell Potential

Native Soil in Fresno County is predominately coarse-grained and will not generally be subject to impactful shrinking or swelling. Results of Atterberg limits tests have a low degree of shrink and swell potential based on plasticity indices less than 18% (Holtz 1959 and USBR 1974). As a baseline, assume that all soils in Fresno County exhibit a low potential for shrink-swell.

Native Soil in Tulare County contains an appreciable proportion of fine-grained material of low to high plasticity. As discussed in Section 6.1.4, all high plasticity soils were encountered with greater frequency in boreholes located south of Deer Creek, as well as S0068R.

Fine-grained soils in Tulare County vary from a low to very high degree of shrink and swell potential.

As a baseline, assume that 80% of fine-grained soil encountered in Tulare County exhibits medium potential for shrink swell, and that 20% exhibits a high-to-very high potential for shrink swell behavior. Further assume that at least half of the fines with high-to-very high shrink swell potential exist at or south of Deer Creek in Tulare County.

High plasticity clays are unsuitable for shallow foundation bearing materials, and where excavations exposure high plasticity clays, remolding and wetting can create difficult working conditions. For bidding purposes, assume all materials of at least high shrink-swell potential must be over-excavated and replaced, to a depth of at least 5 feet.

6.6.6 Collapse and Expansion

In Fresno County, laboratory collapse tests were undertaken on samples from boreholes S0019AR, S0029R, and S0033AR. Samples comprise coarse-grained soils including silty sand and sand from depths of 7.5 feet to 26.5 feet. The results of these tests did not indicate collapsible material.

To assess the susceptibility of fine grained soil to collapse, the liquid limit of 28 samples between 19 feet to 71 feet depth were paired with estimates of dry density made from SPT N-values and moisture content. The data set includes 5 samples in Fresno County and 23 samples from Tulare County. The results are presented in Figure 6.6-1 using criteria derived from Mitchell and Gardner (1975) and Gibbs (1969). The sample from a depth of 65 feet in S0069AR plots as collapsible, and other results in S0030R, S0033AR, and S0069R plots as borderline.

The soils encountered during the GI were not identified as collapsible based on the results shown on Figure 6.6-1. For bidding purposes, assume fine-grained soils of Fresno County are not collapsible. Assume that 1% of the fine gained soils of Tulare County are collapsible.

The potential presence and impact of Dune Sand in Fresno County was discussed in Section 6.1.3. For bidding purposes, assume that collapsible dune sand is present in 5% of the locations in Fresno County, within a depth of 10 feet.

Two bulk samples from S0071R and S0073R in Tulare County plot in Figure 6.6-1 with low and medium expansion potential, respectively. One expansion test, completed on a bulk sample retrieved from 0 to 5 feet depth at borehole S0069AR, resulted in an expansion index of 87.6, or a "high" potential for expansion. In Figure 6.6-1, five additional points fall in the "high" or "very high" range. The high and very high points from S0069R are from 6.5 feet and 5.5 feet depth, indicating that untreated Existing Fill and near-surface clays in this area are prone to undesirable volume changes with changes in moisture content. The remaining three points plotting in the high or very high range are from 65.5 feet to 110.5 feet deep.

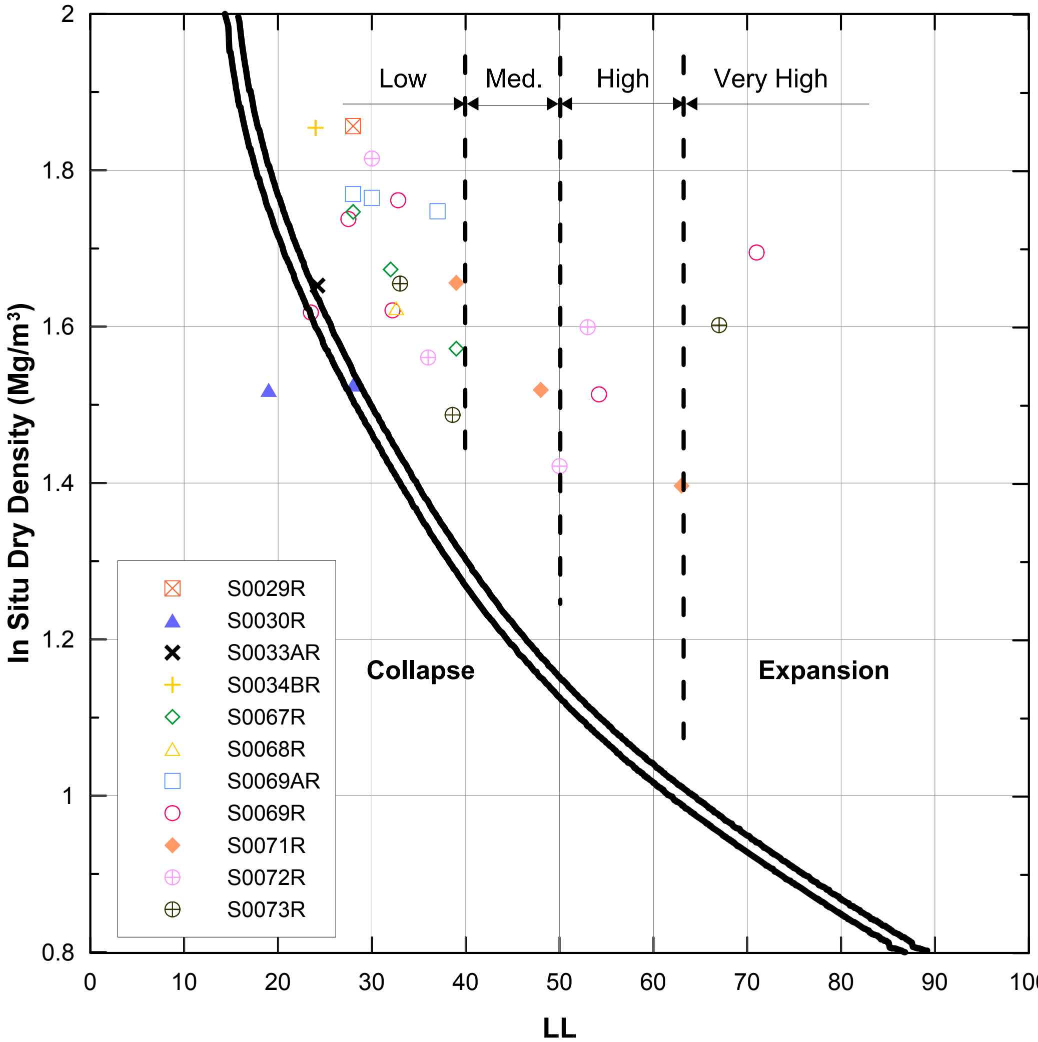

Figure 6.6-1
Collapsibility, Compressibility, and Expansion for Samples with both Liquid Limit and Dry Density Tests (Mitchell and Gardner 1975, and Gibbs 1969)

6.6.7 Land Subsidence

Refer to Section 4.4.4 and the GSHR and GDR for background on potential land subsidence issues in Fresno and Tulare County. Unless directed otherwise by the Scope of Work, for bidding purposes assume that subsidence from groundwater pumping is not an impact to the project area.

6.6.8 Corrosion

For buried concrete and steel elements, Caltrans Corrosion Guidelines (2012) consider a site to be corrosive and/or require further testing if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Resistivity is 1,000 ohm-cm or less.
- Chloride concentration is 500 parts per million or greater.
- Sulfate concentration is 2,000 parts per million or greater.
- pH is 5.5 or less.

Further criteria relevant to corrosion in structural design may be found in the California Building Code and publications by the American Concrete Institute, the American Institute of Steel Construction, and others.

Baseline assumptions for bidding purposes have been provided in Section 6.4.

6.6.9 Long-term settlements

The Existing Fill is expected to be either replaced or recompacted in areas of earthworks where is may contribute to settlements.

The native soil in Fresno County is generally coarse-grained and at least medium-dense, and therefore unlikely to experience appreciable long term settlements.

The native soil in Tulare County is interbedded coarse- and fine-grained material. The fine-grained material is generally stiff to hard, and sufficiently overconsolidated to respond elastically and immediately to the application of new load from embankments. Where fine gained materials may be present in a firm condition, and shallow enough that embankment loads instigate consolidation settlement, it is expected that the drained path to coarse material and the probable construction duration will result in the majority of such settlements being built out prior to placement of permanent track works.

Creep, or secondary settlement, is considered to occur following consolidation of fine-grained material, but can also occur in some coarser-grained materials. In general, secondary settlement associated with the overconsolidated native material underlying embankments is expected to be minor.

For bidding purposes, assume that long-term settlement associated with consolidation and creep in the native soil will not exceed the project design criteria for settlement after construction of permanent way tracks. This baseline presumes a duration of up to a two years from construction of earth embankments and the initial placement of permanent rail track.

In addition, long term settlements may occur due to ground subsidence associated with groundwater extraction as discussed in the Section 6.6.7.

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Section 7.0

Design Considerations

7.0 Design Considerations

7.1 Deep Foundations

Cast-in-drilled-hole (CIDH) piles are planned for the support of most railway structures. Driven piles are planned for most roadway overcrossing bridge abutments. Refer to the PE4P drawings for foundation types at specific locations.

7.1.1 Cast-in-Drilled-Hole Piles

The preliminary design includes deep foundations consisting of cast-in-drilled-hole (CIDH) mono-piles and pile groups to support elevated structures and roadway overcrossings. The selection of CIDH piles was driven by large foundation loads and stringent deflection criteria. Right-of-way constraints and proximity of existing surface structures influenced the preliminary pile type and size selection to those with manageable pile cap footprints.

7.1.2 Axial and Lateral Resistance

Axial resistances of CIDH piles are predominantly determined based on SPT N_{60} values, cone tip resistances, and laboratory undrained shear strengths. Baseline values recommended in Section 6.5 allow for estimating nominal skin friction, end bearing resistance, and p-y curves. Nominal resistances should be determined in accordance with Caltrans amendments to AASHTO requirements as per the HSR Design Criteria Manual.

A significant consideration in the design of deep foundations must be given to lateral load resistance. This resistance is likely to be limited by the stringent deflection criteria necessary to maintain the track-structure interaction criteria. Typical spans and long-span elevated structures will exert large lateral demands on foundations potentially requiring additional piles for lateral resistance, enlarged pile caps, or post-tensioned CIDH piles.

7.1.3 Groundwater

Design of CIDH piles must consider the long-term possibility of groundwater fluctuations. The baseline design groundwater table depth for design of deep foundations is 40 feet in Fresno County and 10 to 30 feet in Tulare County. Perched water may exist at higher elevations, as discussed in Section 8.0.

7.1.4 Downdrag and Uplift Loads

Settlement adjacent to deep foundations can impose downdrag loads. However, soils along the alignment are generally of a consistency and type that is not conducive to time-dependent behavior such as long-term consolidation settlements.

Downdrag loads can also be imposed by collapsible soils and settlements induced by seismic activity, consolidation, or potential localized subsidence. Refer to Sections 4.3.3, 6.6.6, 6.6.7, and 6.6.9 for discussion on possible sources of settlements.

For bidding purposes assume that any settlement of ground adjacent to deep foundations of permanent structure will occur during construction and that long-term downdrag loads will be negligible.

Soils along the CP2-3 alignment are not considered sufficiently expansive to impose uplift loads that require consideration in the design of deep foundations. For the purposes of bidding assume uplift loads due to expansive soils do not need to be considered in the foundation design.

7.1.5 Pile Caps and Abutments

Potential scour at the HSR bridge/viaduct crossings is expected to be in the 15- to 35-foot range for the main channels of the major rivers and creeks for a 100-year storm event, depending on the specific channel, flow, and bridge foundation dimension and configuration at each waterway. Scour countermeasures should be selected, designed, constructed, and maintained per the procedure and methods documented in *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition HEC 23* (FHWA 2009).

7.2 Retaining Walls

7.2.1 Wall Type Selection

Permanent retaining walls for approaches to HSR viaducts include conventional cast-in-place concrete walls and mechanically stabilized earth walls. Mechanically stabilized earth walls are also anticipated at bridge abutments for roadway overcrossings. Mechanically stabilized earth walls shall meet the requirements of Specification Section 31 38 13 Reinforced Slopes and Earth Structures.

7.2.2 Structural Fill

Section 31 05 00 Common Work Results for Earthwork requires that Structural Fill has less than 15% fines. Excavations are required for HSR overcrossings (roadway undercrossings) at Jersey Avenue, SR 43 south of Cross Creek, and Whitney Avenue. Although these excavations are not significant there may be opportunity to recover materials that meet the requirements of Structural Fill.

For bidding purposes assume 15% of the excavated materials will meet Structural Fill requirements where adequate means and methods of separation are employed.

7.2.3 Lateral Deflections

No significant excavations are proposed for the CP2-3 alignment that would pose a threat to existing improvements and structures adjacent to the alignment. Threshold deflection values and response plans associated with excessive deflections will vary by structure and stakeholder requirements, should they become necessary for temporary excavations.

7.2.4 Drainage and Scour

The alignment crosses multiple floodplains including the Kings River, Cross Creek, Tule River, and Deer Creek floodplains. Structures within floodplains should be adequately designed to facilitate drainage.

Adequate drainage is essential to the performance of retaining walls that are not designed for hydrostatic loads. Numerous structures along the alignment have provision for hydraulic crossings. Structural backfill for retaining walls shall be free draining or protected from hydrostatic buildup using geocomposite drainage strips. Additional drainage requirements (apart from the conventional weep holes and toe drains) are not required.

There are several significant bodies of water along the CP2-3 alignment that potentially could require special consideration for scour protection including Coles Slough, Dutch John Cut, Kings River, Cross Creek, and Tule River. Embedded foundations for these structures should consider the potential for scour if located inside the flow boundary. Design of deep foundations for scour

protection shall be in accordance with the procedures provided by Caltrans/the Design Criteria Manual.

Minimum embedment of permanent structure below ground surface shall be in accordance with applicable design standards for the given structure.

7.3 Embankments and At-Grade

7.3.1 Material Selection

Embankment materials consist of embankment fill, transition zone fills, structural fill, drainage layers, and geosynthetics. Embankment materials shall meet the suitability, gradation, and plasticity requirements of Specification Section 31 05 00 Common Work Results for Earthwork. Transition Zone materials are required where embankments support trackway approach structures. Transition Zone materials shall consist of structural fill mixed with cement to meet the strength requirements in Specification Section 31 05 00.

7.3.2 Subgrade Compressibility

Embankment foundation design must consider the potential for post-construction settlement both for static and dynamic conditions. Requirements for overexcavation or other remediation of soft or loose soils should be determined based on characterization of the subgrade and Existing Fill from future GIs to be carried out the Contractor. Typical construction practice for embankment construction in areas of known Existing Fill is to excavate to firm or stable conditions and backfill with material meeting fill and compaction requirements. If firm and stable conditions cannot be reached economically, ground improvement may be necessary.

There are significant zones along the alignment with a potential to encounter Existing Fills that may be 10 to 15 feet deep. Figure 7.3-1 shows a zone between E Manning Avenue and W Mountain View Avenue with numerous shallow depressions within the Dune Sand deposits. These depressions are shown generally striking to the northwest. About 1,000 feet north of HSR intersection with Chestnut, between Davis Ditch and Chicago Ditch, the alignment passes over an in-filled pond. In areas of near-surface granular soils, particle redistribution due to vibration loads (train operation) could be expected over the initial period of railway operation. These areas should be studied to determine whether dynamic recompaction will result in unanticipated deformations after track construction.

The reach from the Kings River Complex to Allensworth is dominated by historical river channels trending typically northeast to southwest. An abundance of relict channels associated with the Kings, Kawaeh, and Tule Rivers can be seen on historical aerial photography and geologic maps. These have all likely in-filled or channelized to facilitate modern agricultural land.

Along the A1 alignment historical ponds and abandoned canals are shown on historical USGS quads between stations 4230+00 to 4270+00, 4320+00 to 4330+00, 4380+00 to 4390+00, and 4455+00 to 4555+00. As noted in the FB GSHR there are numerous small irrigation ponds that can be seen on more recent USGS quads but are not visible on current satellite imagery.

The Contractor should make a complete inventory of these locations by examining all available maps and design a GI to determine their limits.

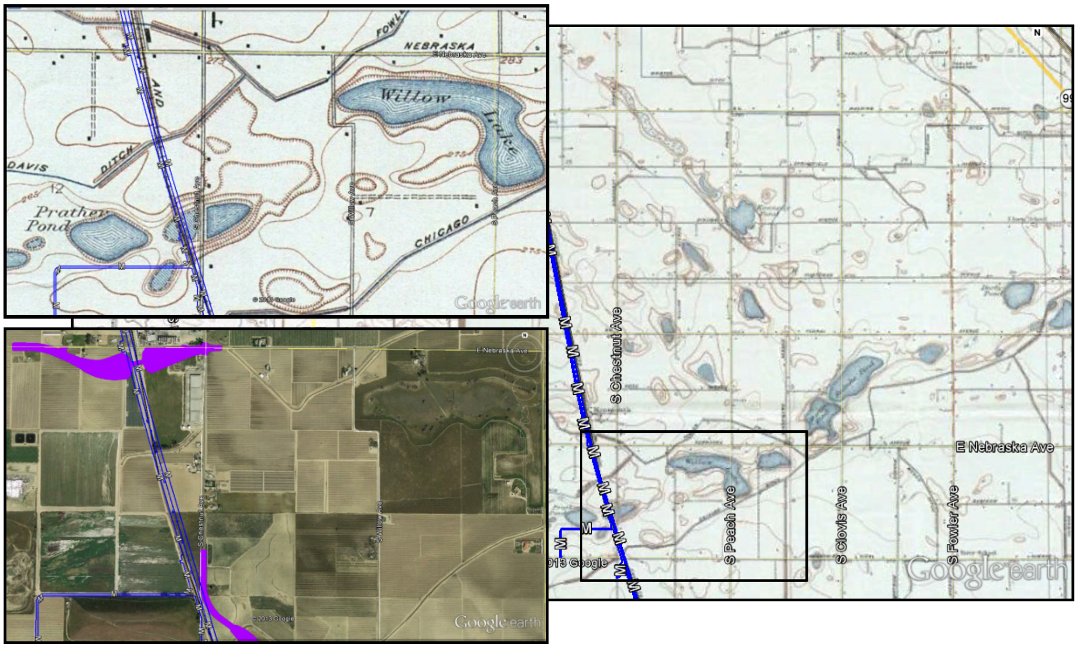

Figure 7.3-1
Historical Ponds (USGS Conejo Quad 1924)

For bidding purposes, assume all Existing Fill is to be removed and replaced with suitable materials in accordance with the Contract Documents unless otherwise directed in the Design Criteria Manual.

7.3.3 Compaction Control

The Contractor shall provide quality control measures to ensure compliance with specified requirements. Embankment foundation and subgrade preparation and the placement and compaction of fills shall be performed under the surveillance of a California-registered Geotechnical Engineer employed by the Contractor, as required by the Contract Documents.

7.3.4 Subgrade Preparation

Subgrade preparation shall meet the requirements of Specification Section 31 05 00. Subgrade preparation includes fine grading, reworking as necessary, and preparation of cut, fill, or embankment upon which the structure and equipment foundations, pipe, subballast, subbase, base, and pavement will be placed. Unsuitable subgrade material, such as weak or compressible soils, shall be removed. The entire surface of subgrade shall be scarified, moisture conditioned, and recompacted in accordance with the Contract Documents. Subgrade stabilization material shall be incorporated if required.

7.3.5 Drainage, Scour, and Erosion

Where an embankment is located in a flood plain, the embankment design shall include slope protection consisting of a drainage layer and protection riprap. The drainage material shall be designed to comply with Terzaghi's filter criteria as defined in the Specification Section 31 05 00. This layer should extend up to the highest flood water level plus additional freeboard as required by the Design Criteria Manual and be underlain by a layer of geosynthetic membrane.

In accordance with the Design Criteria Manual, the highest flood water level is the 100-year flood level. For bidding purposes, a geosynthetic membrane, drainage layer, and rip-rap protection is required for all embankments over 5 feet high within the Kings, Cross Creek, Tule, and Deer Creek floodplains. The approximate limits of the floodplains are provided on Table 7.3-1. Due to the width of the right-of-way and the angle at which the alignment enters the floodplain, the point at which the HSR alignment encounters the FEMA floodplain boundary may vary by several hundred feet across the right-of-way.

Table 7.3-1
Limits of FEMA 100-year Floodplains

Alignment	Floodplain Source	Limits of FEMA 100-yr Floodplain (Stations)
H	Kings River	1486+40 to 1623+50
K4	Cross Creek	2412+40 to beyond end of alignment
C2	Cross Creek	Before start of alignment to 2611+60
C2	Tule River	2858+40 to 3041+70
P	surface ponding	3352+00 to 3432+40
A1	Deer Creek	4006+20 to 4007+60, 4022+00 to 4190+20

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Section 8.0

Construction Considerations

8.0 Construction Considerations

8.1 Regulatory Agencies

If temporary construction dewatering is utilized, a National Pollutant Discharge Elimination System (NPDES) permit from the Central Valley Regional Water Quality Control Board is required. In general, there is a long lead time required to obtain a NPDES permit. Refer to the Contract Documents for Storm Water Pollution Prevention Plan requirements.

Gas detection and monitoring was not in the scope of the preliminary GI. It is the responsibility of the Contractor to investigate potentially gassy conditions that may be present during construction.

Trench excavations, shoring systems, sloped cuts, and other temporary structures shall comply with OSHA 29 CFR 1926.650 and Caltrans regulations.

8.2 Site Constraints

The Contractor shall conduct a site review to identify site specific-constraints that will impact the selection of construction sequence, equipment, and methods. Items affecting the selection of construction means and methods include but are not limited to (1) site accessibility and space restrictions; (2) restrictions on traffic disruption; (3) environmental concerns, including local restrictions on construction noise, vibration, and dust; (4) easement and right-of-way restrictions; (5) railroad operations; (6) watercourses and irrigation infrastructure;(7) relocation(s) of critical area utilities; and (8) location(s) of overhead and underground utilities and nearby structures.

8.3 Corrosive Soils and Groundwater

Both laboratory soil corrosion and groundwater chemistry testing conducted for PE4P design and presented in the CP2-3 GDR indicate the presence of a corrosive subsurface environment.

8.4 Contaminated Soils

The GI conducted for PE4P did not indicate the presence of contaminated soils. However, because the project alignment follows existing freeway and railroad corridors, portions of which are heavily industrialized, the Contractor shall expect to encounter surficially contaminated soils along these corridors during excavation and dispose of them in accordance with all regulatory requirements. No special consideration or baseline is set forth herein.

A soil management plan and site-specific health and safety plan must be implemented prior to initiation of construction activities. If evidence of contaminated soil is found during excavation activities (e.g., stained soil, odors), soil sampling and testing will be required prior to any disposal or reuse. Refer to the Contract Documents for more information.

Abandoned petroleum pipelines exist within and adjacent to the HSR right-of-way between Stations 3015+00 and 3277+00 and 3356+00 and 3437+00 on the C2 and P alignments. A potential for contamination exists at these locations.

8.5 Difficult Excavation

CPTs performed for the PE4P GI occasionally required predrilling at depths where cone penetrometers could not penetrate through hardpan layers. Specific CPT locations and depths where predrilling was required were discussed in Sections 6.5.2.1, 6.5.3.1, and 6.6.2. Near-

surface hardpan layers that might encumber subgrade preparation activities were not encountered.

The Contractor shall expect that excavations for deep foundations will penetrate hardpan layers of variable thickness, hardness, and degree of cementation. Relatively thin, moderately hard, and moderately cemented hardpan layers may be encountered but are not considered to require any specialized drilling equipment.

8.6 Groundwater Inflows

The baseline unconfined groundwater table is below the depth of anticipated excavations made for subgrade preparation; however, due to extensive irrigation that occurs along the entire CP2-3 alignment, there is a potential for perched groundwater to be present during excavation and subgrade preparation operations. The presence of perched groundwater during excavation may reduce the stability of excavated slopes and create unwanted softening or heaving of soils at the base of the excavation.

Very shallow perched groundwater conditions (depths of less than 5 feet) have been observed in excavations made in the vicinity of the alignment. In the event that shallow or perched groundwater conditions exist, appropriate dewatering techniques should be employed. Likely dewatering systems consist of in-excavation sumps. Global dewatering schemes are not anticipated and shall be avoided due to potential impacts on adjacent structures. To the extent practical, permanent retention facilities and other applicable drainage and stormwater facilities should be constructed in the early stages so as to serve as the discharge point for dewatering activities.

8.7 Track and Roadway Subgrade Improvement

Existing Fill was encountered in a number of boreholes along the CP2-3 alignment during the PE4P GI. The Contractor shall anticipate variability in the thickness and suitability of Existing Fill for reuse. Deleterious material in the Existing Fill may include, but is not limited to, wood, glass, brick, metal, coarse gravel, and cobbles. Existing Fill soils are likely suitable for reuse provided they satisfy quality requirements in terms of fines content, gradation, Atterberg limits, and electrochemical properties as required by the Contract Documents.

Soils along the alignment are relatively uniform and possibly suitable for the proposed HSR track construction. However, unsuitable or saturated materials, such as soft clays, loose sands, and existing fills are likely present at shallow depths at some isolated locations in this area. The GI conducted for this design stage is inadequate to characterize the presence and extent of these areas. Some soil improvement measures, such as lime treatment or overexcavation and replacement with engineering fill materials, are likely to be needed to improve the subgrade during the track construction.

8.8 Utilities and Other Obstructions

The Fresno to Bakersfield 15% Record Set Utility Impact Report (URS/HMM/Arup 2013e) identifies 36 High Risk Utilities, numerous Low Risk Utilities, and one Special Utility Consideration. The Contractor is directed to this report for further information on the location and type of utilities at risk.

Existing utility information is provided on the Contract Documents, which include all known utilities such as the following:

- Overhead high-voltage transmission main relocations.
- Buried longitudinal utilities within freight rail rights-of-way where the freight rail trackage requires relocation to accommodate the HSR right-of-way.
- Gas mains.
- Fiber optic lines.

8.9 Deep Foundations

Deep foundations will be required to support the viaduct piers, retaining walls, and bridge abutments. There are a number of different issues that should be considered regarding deep pile foundations that are dependent on the type of pile being installed. The anticipated deep foundation types for this project include CIDH piles and driven piles.

8.9.1 Driven Piles

Due to the presence of very dense sand/silty sand layers at various depths throughout the project site, hard driving conditions may be encountered during installation of driven concrete piles. Piles may be subject to refusal if either the soil is too dense to accept the pile or the hammer energy is too low to drive the pile. The Wave Equation Analysis of Piles (WEAP) can be used to help select the proper pile driving equipment and predict drivability of piles. WEAP simulates and analyzes the dynamics of a pile under hammer impacts according to one-dimensional elastic wave propagation theories. The results are used to predict the dynamic compatibility of the hammer-pile soil for evaluation of drivability of driven piles.

The Contractor shall select equipment to safely install the pile to the desired depth and capacity without damage. As per Section 49-1.05 of the Caltrans Standard Specifications, undersize predrilling can be used to facilitate the pile driving in thick and dense sand layers. Predrilling holes shall not be greater than the least dimension of the piles. In addition, driven steel pile (open-ended pipe pile or H pile) can also be considered to penetrate layers with difficult driving conditions.

Baselines for difficult driving conditions are not set forth herein.

8.9.2 Cast-in-Drilled-Hole Piles

CIDH piles can be achieved in this region by a number of techniques, which include drilling an open dry hole, drilling the hole with water, drilling the hole with a bentonite slurry, and drilling a temporarily cased hole. Each of these methods has its advantages and disadvantages. For baseline purposes, assume CIDH piles will require temporary support to prevent caving given the granular nature of the soils.

Cobbles and boulders can impede drilling operations. Cobbles and boulders were not encountered during the exploration. Discussion and baseline statements on potential debris/obstructions in fill and hardpan and cemented soil conditions were provided in Sections 6.1.2, 6.5.2.1, 6.5.3.1, and 6.6.2.

8.10 Excavations

Shallow excavations will be required for the pile caps, footings, and subgrade preparation of at-grade and retained areas. Trenching may also be required for utility installation. For the shallow depth of these excavations, excavations may be cut vertically if the soils will "stand-up" without shoring but only within the limits prescribed by OSHA and only under the supervision of a "competent person" as defined by OSHA and/or Cal/OSHA.

In some areas the soils may be too loose or granular to achieve a 5-foot excavation and a sloped cut or bracing must be used in conjunction with falsework and engineered backfill. Backfill at sloped pile cap excavations must be compacted to provide sufficient lateral resistance.

Surface runoff on the site should be controlled so that it does not flow into open excavations. Surface runoff shall conform to standard Storm Water Pollution Prevention Plan requirements.

8.11 Existing Features

Existing features along the CP2-3 alignment of interest include the following:

- BNSF Railroad.
- SR 43.
- Kings River Complex.
- Kings County Landfill.
- Baker Commodities Rendering Facility.
- Ponderosa Community.
- Lakeside Cemetery.
- SJV Railroad.
- SR 198.
- Cross Creek.
- Kaweah Conservation District Mitigation Area.
- Slayer Farms Airport.
- Tule River.
- Deer Creek.
- Stoil Railroad Spur.
- Numerous irrigation canals.

8.12 Environmental Concerns

Noise and vibrations produced through the construction of the project structures should adhere to the project environmental management plan and comply with state and federal health and safety regulations.

Construction schedules shall consider earthwork to take advantage of the dry season (April through October). Earthwork in the dry season must include provisions for dust mitigation in accordance with local and regional air quality regulations. Dust in the SJV is known to contain spores that cause Valley Fever. Dust control will be of paramount importance.

Requirements for erosion control are found in Specification Section 31 05 00. Other environmental concerns may be found in the FB EIR/EIS.

8.13 Archeological and Historic Environmental Resources

As a result of the studies conducted in support of the FB EIR/EIS, seven archaeological sites were identified within the project alignments. None of these sites were considered significant and thus do not warrant additional treatment or mitigation (see *California High-Speed Train Fresno to Bakersfield Archaeological Survey Report* and *California High-Speed Train Fresno to Bakersfield Supplemental Archaeological Survey Report*). However, due to limitations in permission to enter, only approximately 20% of the HSR project alignment footprint has been subject to archaeological survey. In addition, a number of areas were identified that will require additional investigations and potentially require monitoring during construction, such as the area surrounding Alpaugh (see Chapter 3.17 of the EIR/EIS). These future studies will be conducted

per the stipulations of the Section 106 Programmatic Agreement and the Archaeological Treatment Plan and Memorandum of Agreement. These documents will define the process by which these treatment measures will be applied to each known resource and will outline measures for the phased identification of historic properties as additional parcel access is obtained and design work is completed.

A number of significant historic architectural resources have been identified within the HSR project footprint (see *California High-Speed Train Fresno to Bakersfield Historic Architectural Survey Report*, *California High-Speed Train Fresno to Bakersfield Historic Property Survey Report*, and supplements prepared in 2013 and 2014). As with archaeological resources, in addition to the mitigation measures provided in the EIR/EIS, a series of treatment measures will be formulated per the stipulations of the Section 106 Programmatic Agreement and the Built Environment Treatment Plan and Memorandum of Agreement. These documents will define the process by which these treatment measures will be applied to each known resource and will outline measures for the phased identification of historic properties as additional parcel access is obtained and design work is completed.

8.14 Geotechnical Permitting

Geotechnical explorations must be conducted during the design-build phase of the project to augment the geotechnical data collected during PE4P. Geotechnical exploration permitting generally falls in two categories: (1) permits to drill within riparian areas and (2) permits to drill outside riparian areas. Drilling permits for areas outside of riparian habitat are typically obtained from city and county environmental health agencies.

Permits to encroach on jurisdictional rights-of-way should be obtained from the local agency, county, or Caltrans, as appropriate.

8.15 Construction Consideration Matrix

Table 8.15-1 has been prepared to capture the site conditions that would be of concern to a bidding contractor, from an engineer’s perspective. The list is not exhaustive but identifies some conditions at each of the planned structures that could have cost implications when considered as part of the bid preparation.

Table 8.15-1
Construction Considerations Matrix

Location	Approximate Start Station (ft)	Approximate End Station (ft)	Track Subgrade Improvement	Hydraulic Crossing	Shallow Groundwater	Difficult excavation - Hardpan	Buried Utilities and Other Obstructions	Compressible Soil	Expansive Soil	Collapsible Soil	Erodible Soil	Potential for Flooding	Corrosive Soil	Regulatory Agencies	Deep Foundations	Protection of Existing Structures/Utilities	Unforeseen Ground Conditions	Need for Additional Geotechnical Data
At-Grade	577+60	1086+00	X	X	X		X			X	X			X		X	X	X
Overcrossings				X	X	X	X				X		X	X	X	X	X	X
Retained	1086+00	1105+70	X				X			X	X						X	X
Aerial	1105+70	1156+20					X			X	X			X	X	X	X	X
Retained	1156+20	1173+50	X								X					X	X	X
At-Grade	1173+50	1439+19	X	X	X	X					X		X			X	X	X
Overcrossings						X	X				X		X		X		X	X
Retained	1439+19	1463+58	X	X	X	X			X		X					X	X	X
Aerial	1463+58	1596+56		X	X		X		X				X	X	X	X	X	X
At-Grade	2883+63	2966+50	X	X	X		X									X	X	X
Retaining Wall	2966+50	2989+36	X		X			X									X	X
Aerial Structure	2989+36	3046+02		X	X		X	X	X		X		X	X	X	X	X	X
Retaining Wall	3046+02	3064+70	X		X		X	X	X		X						X	X
At-Grade	3064+70	3982+20	X	X	X		X	X	X		X	X		X		X	X	X
Overcrossings				X	X		X	X	X		X	X	X		X		X	X
Retaining Wall	3982+20	4005+25	X	X	X			X	X		X						X	X
Aerial Structure	4005+25	4067+65	X	X	X		X	X	X		X		X	X	X	X	X	X
Retaining Wall	4067+65	4085+95	X		X			X	X		X						X	X
At-Grade on Fill	4085+95	4435+50	X	X	X	X	X	X	X	X		X		X			X	X
Overcrossing					X	X	X		X	X		X	X		X		X	X

Section 9.0

Instrumentation and Monitoring

9.0 Instrumentation and Monitoring

The design criteria mandate specific limits on post-construction total and differential settlement of embankments, transition zones, and abutments that will require accurate measurements be made. Moreover, subsidence rates along the alignment are continuing. Thus, establishing an early array of surface settlement monuments and a periodic monitoring program early in the contract to verify the subsidence rates could be a critical element of the Contractor's design. Refer to the contract documents for specific instrumentation and monitoring requirements.

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Section 10.0

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Section 11.0

Glossary

11.0 Glossary

Atterberg limits: The water contents of a soil mass corresponding to the transition between a solid, semi-solid, plastic solid, or liquid. Laboratory test used to distinguish the plasticity of clay and silt particles.

Boulder: A rock fragment that will not pass through a 12-inch (305-millimeter) square opening, no matter how it is oriented in the opening. Boulder sizes are defined by the smallest size opening that the boulder can be oriented to pass through.

Bulking/swell factor: $\frac{\text{volume of soil after excavation}}{\text{volume of soil in situ}}$ (volume of soil after excavation) / (volume of soil in situ).

Cobbles: Soil particles between 3 inches (76 millimeters) and 12 inches (305 millimeters) in size.

Cohesion: The force that holds together molecules or like particles within a substance.

Cohesionless soils: Granular soils (silt, sand, and gravel type) with no shear strength unless confined.

Cohesive soils: Contains clay minerals and possesses plasticity.

Consolidation: Reduction in soil volume due to squeezing out of water from the pores as the soil comes to equilibrium with the applied loads.

Dewatering: The removal of groundwater to reduce the flow rate or diminish water pressure. Dewatering is usually done to improve conditions in surface excavations and to facilitate construction work.

Dry **u**nit **w**eight: The weight of solids (soil grains) to the total unit volume of soil. Units lb/ft^3, kN/m^3.

Firm, firm ground: Soil that remains stable in walls and face of an opening without initial support for sufficient time to permit installation of final support.

Flowing, flow, flowing ground: Soil that moves like a viscous liquid into an excavation.

Grain size distribution, particle size distribution: Soil particle sizes that are determined from a representative sample of soil that is passed through a set of sieves of consecutively smaller openings.

Groundwater: Water that infiltrates into the earth and is stored in the soil and bedrock within the zone of saturation below the earth's surface.

Hydrostatic head, hydrostatic pressure, pressure head: The height of a column of water required to develop a given pressure at a given point. Head may be measured in either height (feet or meters) or pressure (pounds per square inch, kilograms per square centimeter, or bars).

Natural water content: The ratio between the mass of water and the mass of soil solids. w = (total unit weight – dry unit weight) / dry unit weight.

Normalized cone resistance (Q_t): CPT tip resistance in a non-dimensional form and taking account of the in situ vertical stresses.

$$Q_t = (q_t - \sigma_{v0})/\sigma_{v0}'$$

Normalized friction ratio (F_r): The ratio, expressed as a percentage, of the sleeve friction (f_s) to the cone resistance (q_t) taking account of the in situ vertical stresses.

$$F_r\,(\%) = \left(\frac{f_s}{q_t - \sigma_{v0}}\right) 100 F_r\,(\%) = [f_s / (q_t - \sigma_{vo})]100$$

Normalized CPT soil behavior type (SBT_N): Soil behavior type based on normalized cone resistance (Q_t) and normalized friction ratio (F_r).

Normally consolidated: A soil where the current effective overburden pressure is equal to the maximum overburden pressure.

Perched groundwater: An unconfined groundwater body in a generally limited area above the regional water table and separated from it by a low-permeability, unsaturated zone of bedrock or soil.

Permeability: The capacity of bedrock or soil to permit fluids to flow through it.

q_c: CPT cone resistance.

q_t: CPT cone resistance corrected for pore water effects, where A_n is the cone tip area ratio:

$$q_t = q_c + u_2(1 - A_n)$$

Raveling, slow raveling, fast raveling: Chunks or flakes of material drop out of the excavated surface due to loosening or to overstress and "brittle" fracture. In fast raveling ground, the process starts within a few minutes; otherwise, the ground is slow raveling.

Regional subsidence: Large-scale, slow-occurring, typically unnoticeable deformation of the ground surface attributable to tectonic activity, groundwater abstraction, or extraction of other liquids or gasses. Typical magnitudes of regional subsidence are on the order of inches or feet occurring over decades across tens of miles.

Running, cohesive running ground: Granular soils that move freely into the excavated area. Granular materials without cohesion are unstable at a slope greater than their angle of repose. When exposed at steeper slopes, they run like granulated sugar or dune sand until the slope flattens to the angle of repose. Cohesive running ground exhibits some apparent cohesion that exists from moisture content, weak cementation, and overconsolidation.

Shear strength: The maximum shear stress that a soil can sustain under a given set of conditions. For clay, shear strength = cohesion. For sand, shear strength = the product of effective stress and the tangent of the angle of internal friction.

Shrinkage factor: (volume of soil after compaction) / (volume of excavated soil before compaction)

$$\frac{\text{volume of soil after compaction}}{\text{volume of excavated soil before compaction}}$$

Specific gravity: The ratio of the density of a body or a substance to the mass of an equal volume of water.

Standard penetration test, N-value: Field test performed in general accordance with ASTM D 1586, Test Method for Penetration Test and Split – Barrel Sampling of soils. Test involves driving a 2-inch OD, 1.375 inch ID, split spoon sampler with a 140-pound hammer, falling freely from a height of 30 inches. The number of blows required to achieve each of three 6-inch increments of sampler penetration is recorded. The density of cohesionless or coarse-grained soils, and relative consistency of cohesive or fine-grained soils is defined as below:

Cohesionless Soils		Cohesive Soils	
N, SPT Blows/ft	**Relative Density**	**N, SPT Blows/ft**	**Relative Consistency**
0–4	Very loose	Under 2	Very soft
4–10	Loose	2–4	Soft
10–30	Medium dense	4–8	Medium stiff
30–50	Dense	8–15	Stiff
Over 50	Very dense	15–30	Very stiff
		Over 30	Hard

Structural fill: Soils used as fill, such as retaining wall backfill, foundation support, dams, and slopes, that are to be placed in accordance to engineered specifications. These specifications may delineate soil grain-size, plasticity, moisture, compaction, angularity, and many other index properties depending on the application.

Swelling, swelling ground: Soil that undergoes a volumetric expansion resulting from the addition of water. Swelling ground may appear to be stable when exposed, with the swelling developing later. Ground absorbs water, increases in volume, and expands slowly into the tunnel. Increase in soil volume; volumetric expansion of particular soils due to changes in water content.

Total Unit Weight: Ratio between the total weight of soil including water and the total volume of the soil.

u_2: Pore pressure generated during cone penetration and measured by a pore pressure sensor just behind the cone.

Unconsolidated: Loose sediment, lacking cohesion or cement.

Unified Soil Classification System (USCS): A system of soil classification based on grain size, liquid limit, and plasticity of soils.

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Appendix A
Soil Parameter Interpretations

Table of Contents

Page

Tables

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Figures

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A1.0 Introduction

This appendix presents the results of data analyses undertaken to assist development of the baseline soil parameters presented in Section 6 of the main report.

The purpose of this appendix is to present the variability of baselined soil properties and parameters associated with the ground conditions encountered during the ground investigation. Histograms and cumulative distributions have been prepared to present the range, mean, median, and standard deviation of data collected during this ground investigation. These interpretations are provided to illustrate the uncertainty associated with the estimates of baseline soil parameters.

The appropriateness of the data presented herein have been reviewed, and in some cases, outlier data was excluded from interpretations. Correlations used to estimate soil parameters have been restricted to maximum values considered reasonable based on engineering judgment.

Soil parameters have been measured and interpreted following TM 2.9.10 *Geotechnical Analysis and Design Guidelines*, in general accordance with Geotechnical Engineering Circular No. 5 (FHWA 2002) and AASHTO LRFD Bridge Design (2010) recommendations.

Cone penetration test (CPT) interpretations were based primarily on correlations presented in Lunne (1997). In addition, CPT data collected during the investigations was analyzed using the commercially available software CPeT-IT v1.7.6.42, developed by Geologismiki.

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A2.0 CPT and Drilling Correlations

A2.1 Total Unit Weight

A2.1.1 CPT Correlation

Total unit weight was estimated from CPT results using the following correlation presented in Lunne (1997):

Table A2.1-1
Unit Weight by SBT, from CPT data

SBT [a]	SBT description	Unit Weight, γ_t (psf)
1	Sensitive fine grained	111.4
2	Organic soil	79.6
3	Clay	111.4
4	Silty clay to clay	114.6
5	Clayey sitl to silty clay	114.6
6	Sandy silt and clayey silt	114.6
7	Silty sand and sandy silt	117.8
8	Sand and silty sand	120.9
9	Sand	124.1
10	Sand to gravelly sand	127.3
11	Very stiff fine grained [b]	130.5
12	Sand to clayey sand [b]	120.9

[a] SBT uses an earlier interpretive method for soil behavior type by Robertson et al (1986). Note that the main report often referes to SBT_N, a normalized method developed by Robertson (1990) and revised (2010).
[b] heavily overconsolidated and/or cemented

A2.2 Undrained Shear Strength

A2.2.1 CPT Correlation

Undrained shear strength was estimated from CPT results using the following correlation presented in Lunne (1997):

$$s_u = \frac{q_c - \sigma_{vo}}{N_k}$$

Where:

q_c = Measured cone resistance

σ_{vo} = (Total) Vertical overburden stress

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

N_k = Cone factor; taken as 17 for non-fissured overconsolidated clay

A2.3 Effective Friction Angle

A2.3.1 CPT Correlation

Effective fiction angle was estimated from CPT results using the following correlation presented in FHWA GEC No. 5 (after Robertson, 1983):

$$\phi' = \arctan\left[0.1 + 0.38\log\left(\frac{q_t}{\sigma'_{vo}}\right)\right]$$

Where:

σ'_{vo} = Effective vertical overburden stress

$q_t = q_c + u_2(1-a)$ = Corrected cone resistance

u_2 = Pore pressurement measreument behind cone

a = Net cone area ratio (0.80 for site equipment used)

A2.3.2 Drilling Correlation

Effective fiction angle was estimated from SPT results using the following correlation presented in FHWA GEC No.5 (after Hatanaka and Uchida, 1996):

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20°$$

Where:

$(N_1)_{60}$ = SPT N-value corrected for overburden and field procedures (Section A2.4.2)

A2.4 Standard Penetration Test Blow Count

A2.4.1 CPT Correlation

SPT N_{60} was estimated from CPT results using the following correlation used in CPeT-IT v1.7.6.42:

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2917 \cdot I_c}}$$

Where:

I_c = Soil Behavior Type Index

Given By:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$$

Where:

Q_t = Normalized cone penetration resistance

F_r = Normalized Friction Ratio

A2.4.2 Drilling Correction

The SPT correction for field procedures (energy) was applied as follows:

$$N_{60} = C_E N_{SPT}$$

Where:

N_{SPT} = Uncorrected field SPT N-value. Where a modified California sampler was used, the following correlation was used: $N_{SPT} = 0.64 N_{MC}$

C_E = Correction factor for Energy Ratio (ER) as measured in the field = ER/60

The SPT correction for overburden was applied as follows:

$$(N_1)_{60} = C_N N_{60}$$

Where:

N_{60} = SPT N-value corrected for hammer energy

C_N = Stress normalization parameter calculated as $C_N = \left[0.77 \log\left(\frac{40}{\sigma'_v}\right)\right] \leq 2.0$

A2.5 Cone Tip Resistance

The measured cone resistance used for the statistical analyses refers to the static cone resistance q_c recorded during cone penetration testing, as follows:

$$q_c = \frac{Q_c}{A_c}$$

Where:

Q_c = Force acting on the cone

A_c = Projected area of the cone

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

A2.6 Soil Modulus

A2.6.1 CPT Correlation

For coarse-grained material, soil modulus was estimated from CPT results using the following correlation (after AASHTO 2010):

$$E_s = 4q_c$$

For fine-grained material, soil modulus was estimated from undrained shear strength using the equation below.

$$E_u = 300s_u$$

Where:

E_u = Undrained soil modulus of fine grained soil

s_u = Undrained shear strength, estimated from CPT data as per Section A2.2

A2.6.2 Drilling Correlation

For coarse-grained material, soil modulus was estimated from SPT results using the elastic constant for Category 2, indicated in Table A2.6-1 (after AASHTO 2010). .

Table A2.6-1
SPT Correlation to Soil Modulus by Soil Type

Category	Soil Type	Soil Modulus (tsf)
1	Silt, sandy silts, slightly cohesive mixtures	$4(N_1)_{60}$
2	Clean fine to medium sands and slightly silty sands	$7(N_1)_{60}$
3	Coarse sands and sands with little gravel	$10(N_1)_{60}$
4	Sandy gravels and gravels	$12(N_1)_{60}$

Fine grained soils generally comprise very stiff overconsolidated mixtures of clay and silt, and are not applicable to Category 1. Estimation of soil modulus for fines using SPT N was not undertaken. Refer to CPT correlation above and further discussion in the main report.

A3.0 Fresno County

The following sections present the results of statistical analysis performed on data obtained from boreholes and CPTs within Fresno County.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in two layers: (1) upper 25 feet of soils (excluding Existing Fill) and (2) soils below 25 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, drilling, or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A3.1 Total Unit Weight

Table A3.1-1
Statistical Summary of Total Unit Weight – Fresno County

Total Unit Weight	CPT		Drilling*	
	Upper 25 ft	Below 25 ft	Upper 25 ft	Below 25 ft
No. Tests	1597	5863	17	45
Mean, pcf	121.2	122.2	111.3	119.6
Median, pcf	121.0	124.0	111.8	119.3
Standard Deviation, pcf	3	3	8	8
Maximum, pcf	131	131	129.0	134.9
Minimum, pcf	111	111	101.3	101.6

* Unit weight from drilling determined from samplers with full recovery.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

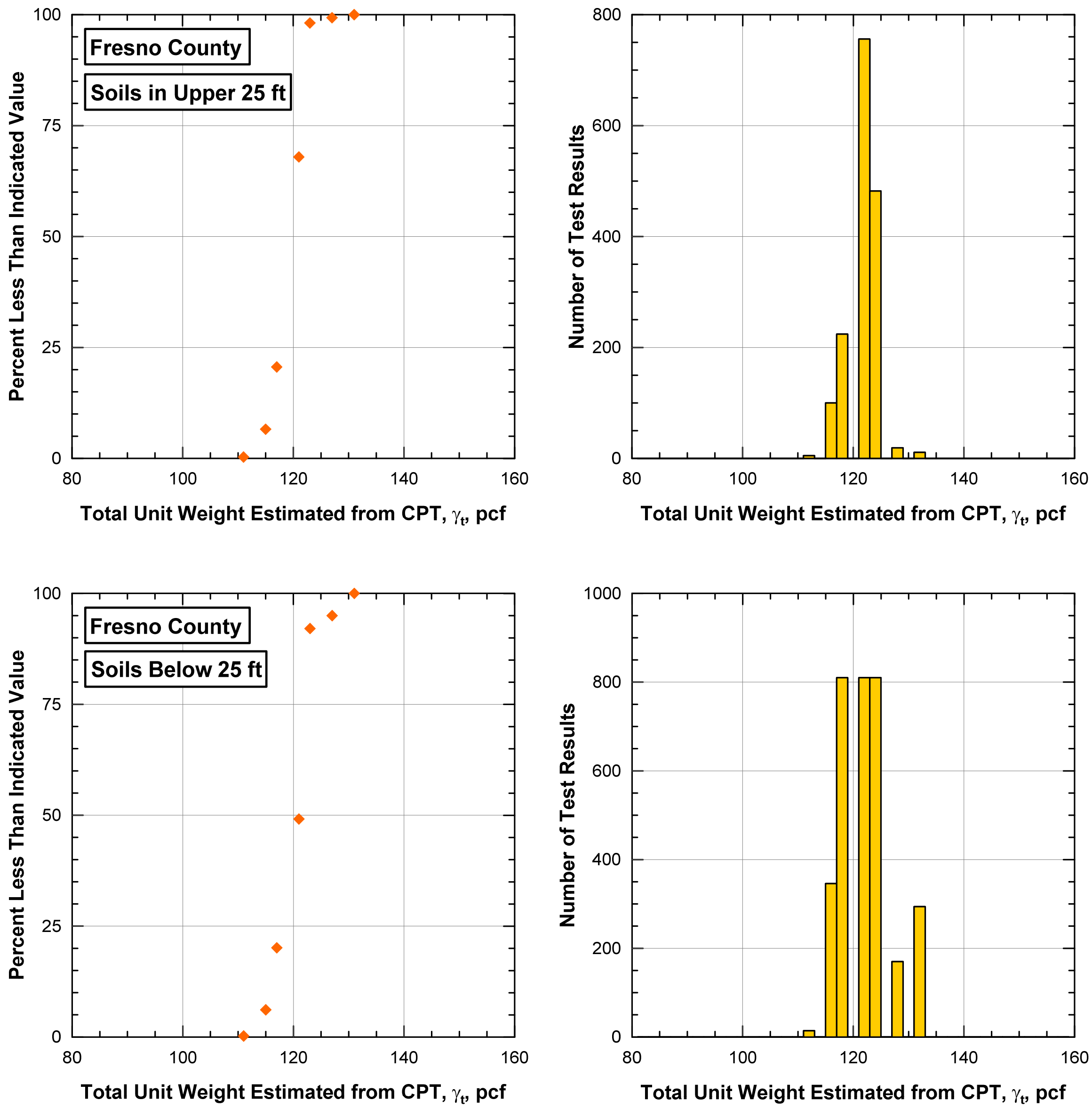


Figure A3.1-1
Statistical Summary of Total Unit Weight Estimated from CPT – Fresno County

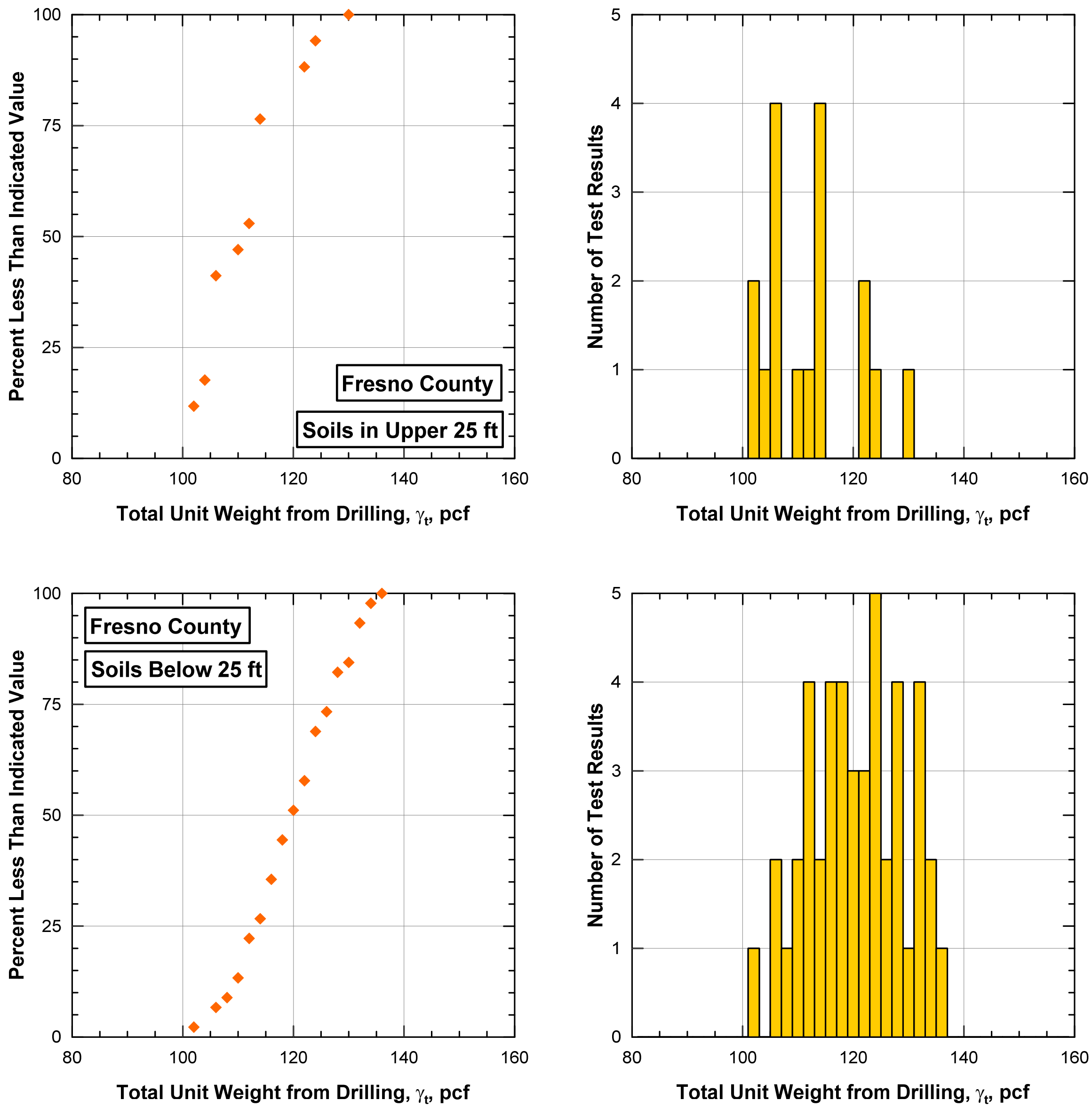
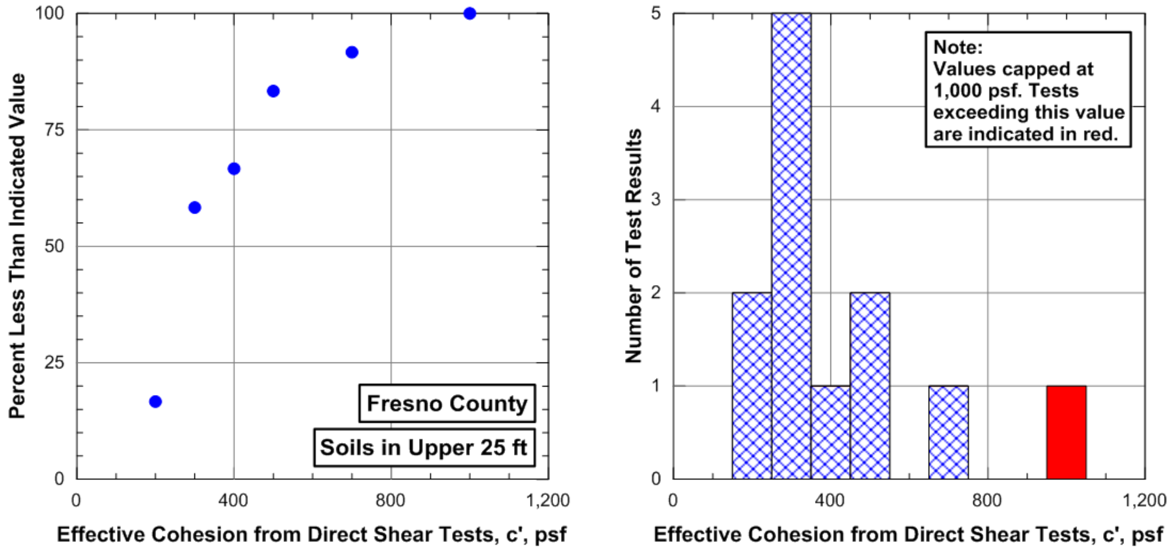


Figure A3.1-2
Statistical Summary of Total Unit Weight Estimated from Drilling – Fresno County

A3.2 Effective Cohesion

Table A3.2-1
Statistical Summary of Effective Cohesion – Fresno County

Effective Cohesion	Laboratory	
	Upper 25 ft	Below 25 ft
No. Tests	12	9
Mean, psf	379	554
Median, psf	275	430
Standard Deviation, psf	252	358
Maximum, psf	1000	1000
Minimum, psf	140	100



Note: Effective cohesion and effective friction angle must be interpreted collectively.

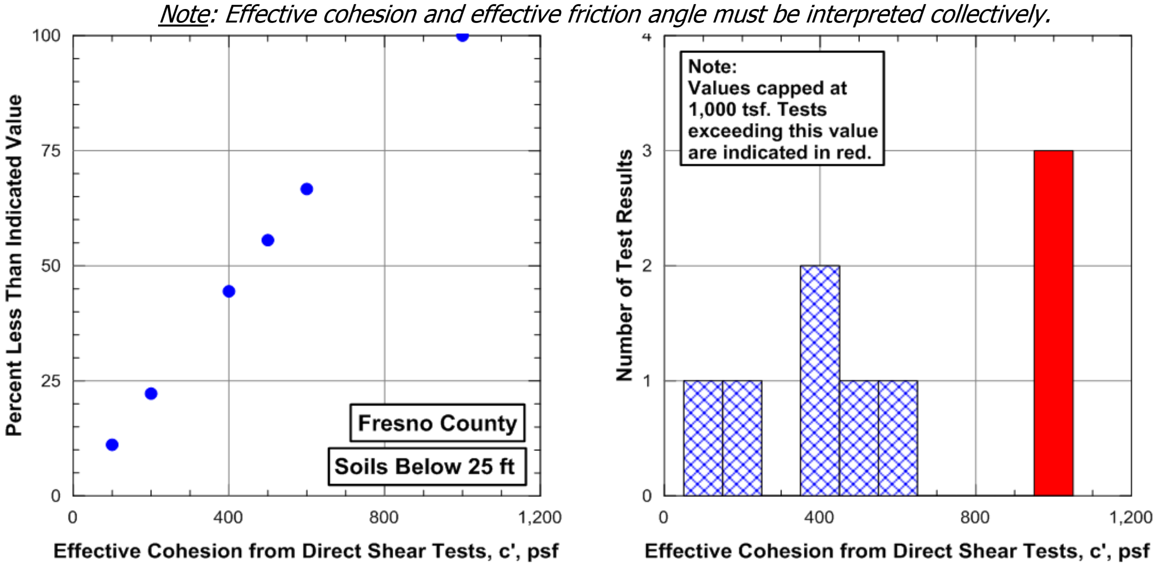
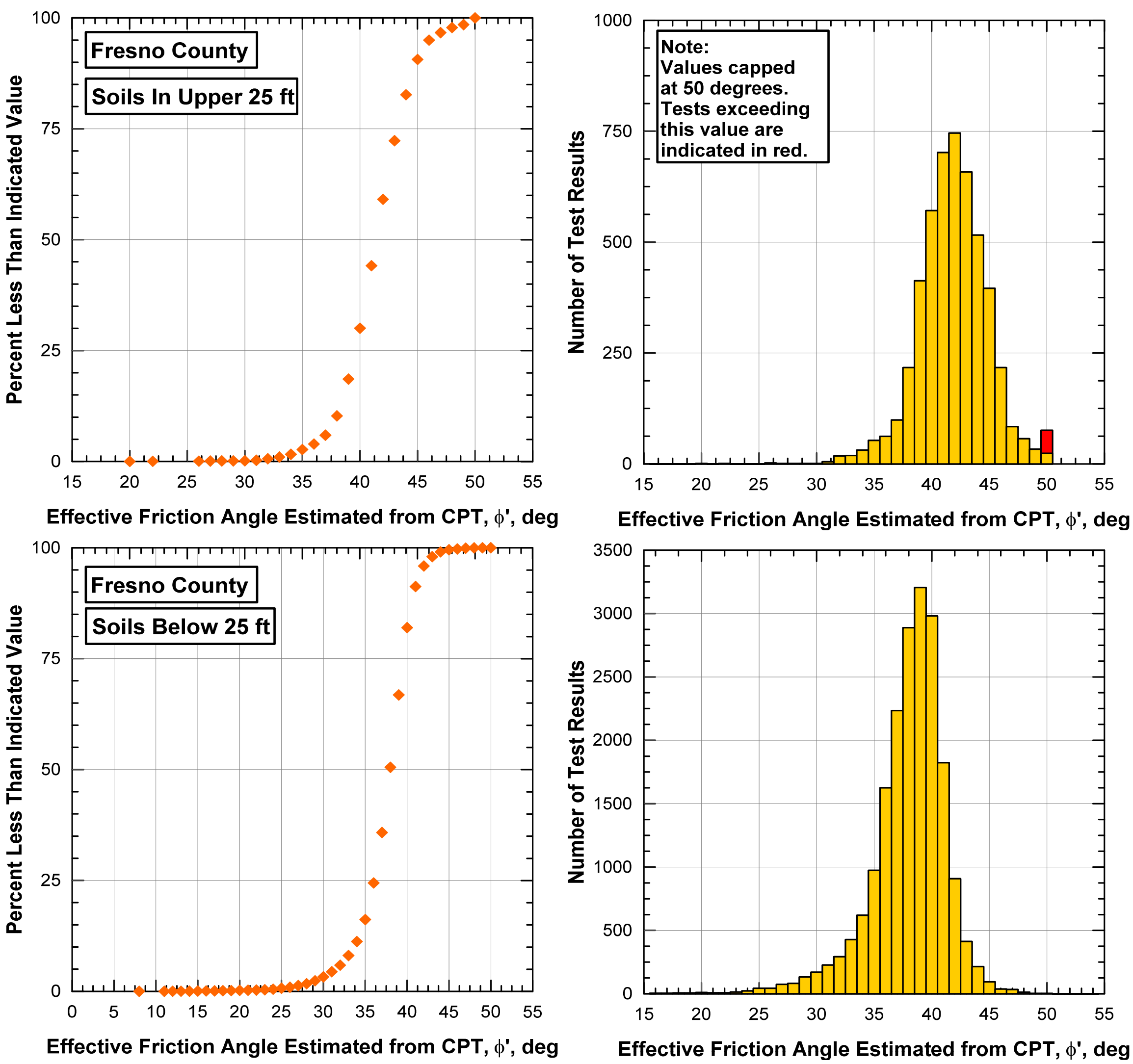


Figure A3.2-1
Statistical Summary of Effective Cohesion from Laboratory Results – Fresno County

A3.3 Effective Friction Angle

Table A3.3-1
Statistical Summary of Effective Friction Angle – Fresno County

Effective Friction Angle	CPT		Drilling		Laboratory	
	Upper 25 ft	Below 25 ft	Upper 25 ft	Below 25 ft	Upper 25 ft	Below 25 ft
No. Tests	4981	19657	39	150	15	11
Mean, deg	41	37	41	47	35	33
Median, deg	41	38	41	48	34	34
Standard Deviation, deg	3	3	4	3	4	8
Maximum, deg	50	49	50	50	50	43
Minimum, deg	19	8	35	30	30	10



Note: Effective cohesion and effective friction angle must be interpreted collectively.

Figure A3.3-1
Statistical Summary of Effective Friction Angle Estimated from CPT Data– Fresno County

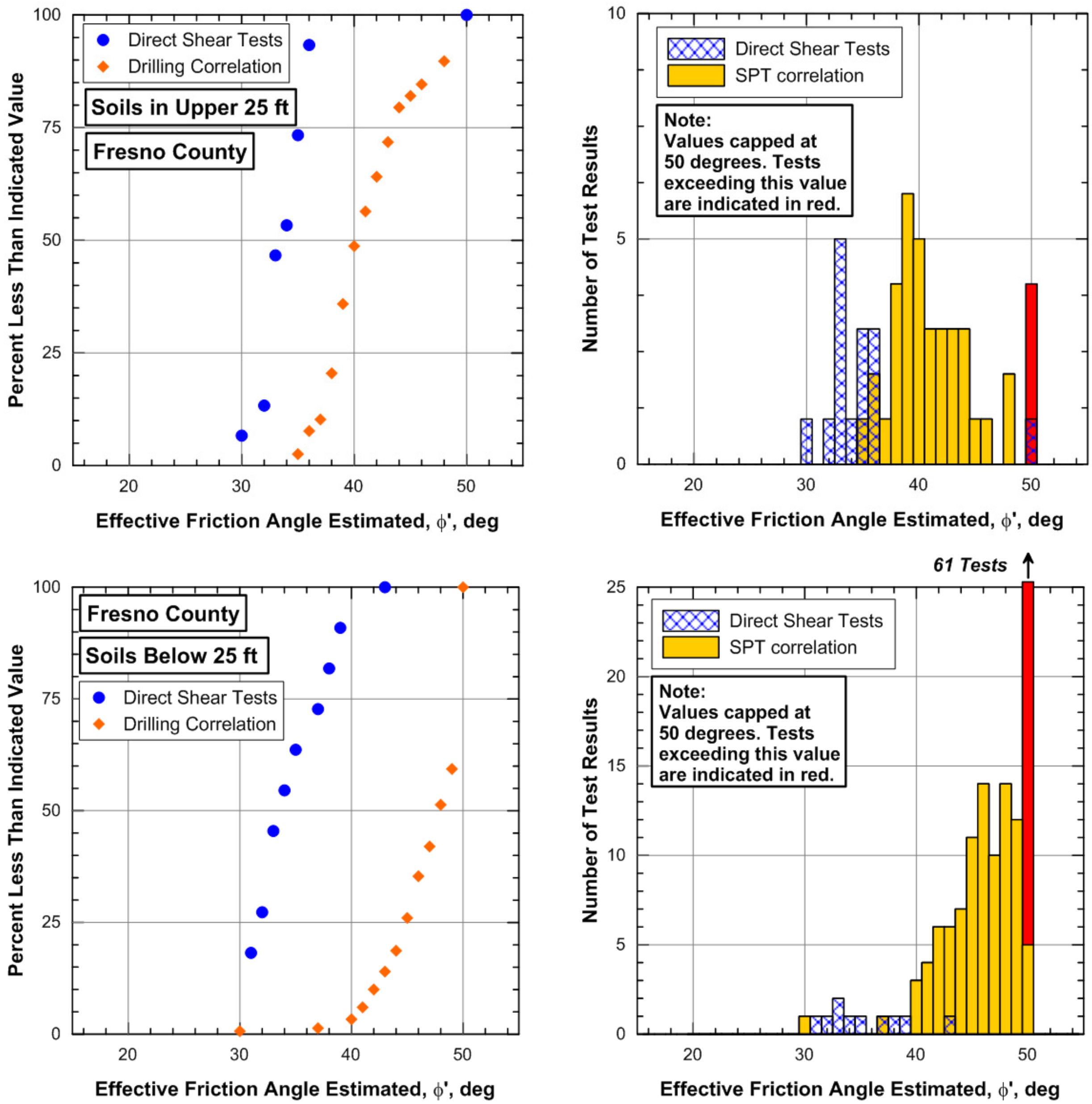


Figure A3.3-2
Statistical Summary of Effective Friction Angle Estimated from Drilling and Laboratory Data – Fresno County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

A3.4 SPT N_{60}

Table A3.4-1
Statistical Summary of SPT N60 – Fresno County

SPT N60	CPT		Drilling	
	Upper 25 ft	Below 25 ft	Upper 25 ft	Below 25 ft
No. Tests	4981	19656	36	150
Mean, blows/ft	26	54	23	63
Median, blows/ft	22	51	18	58
Standard Deviation, blows/ft	15	19	15	23
Maximum, blows/ft	99	99	99	99
Minimum, blows/ft	4.0	5.0	10	6

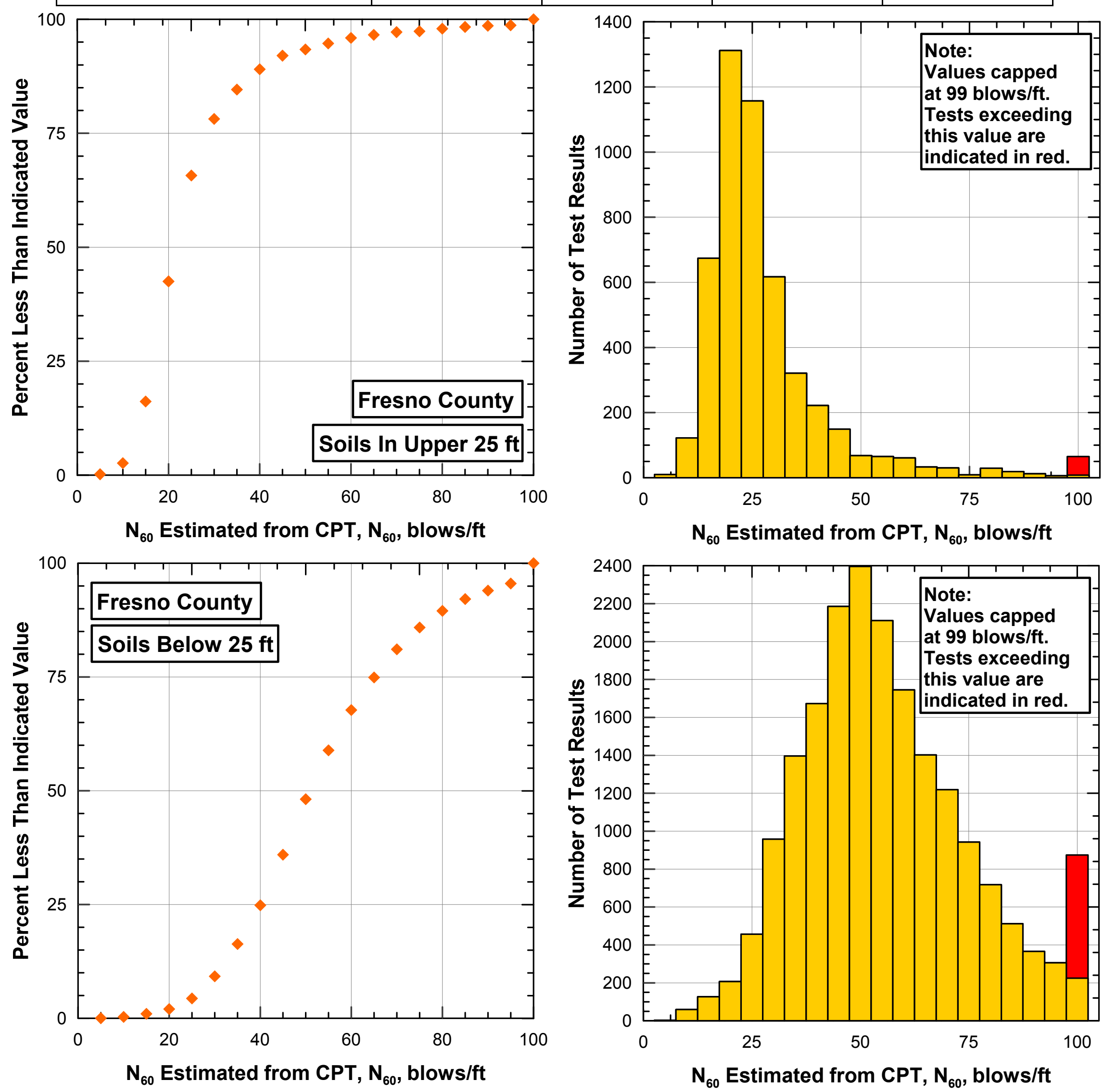


Figure A3.4-1
Statistical Summary of N_{60} Estimated from CPT Data – Fresno County

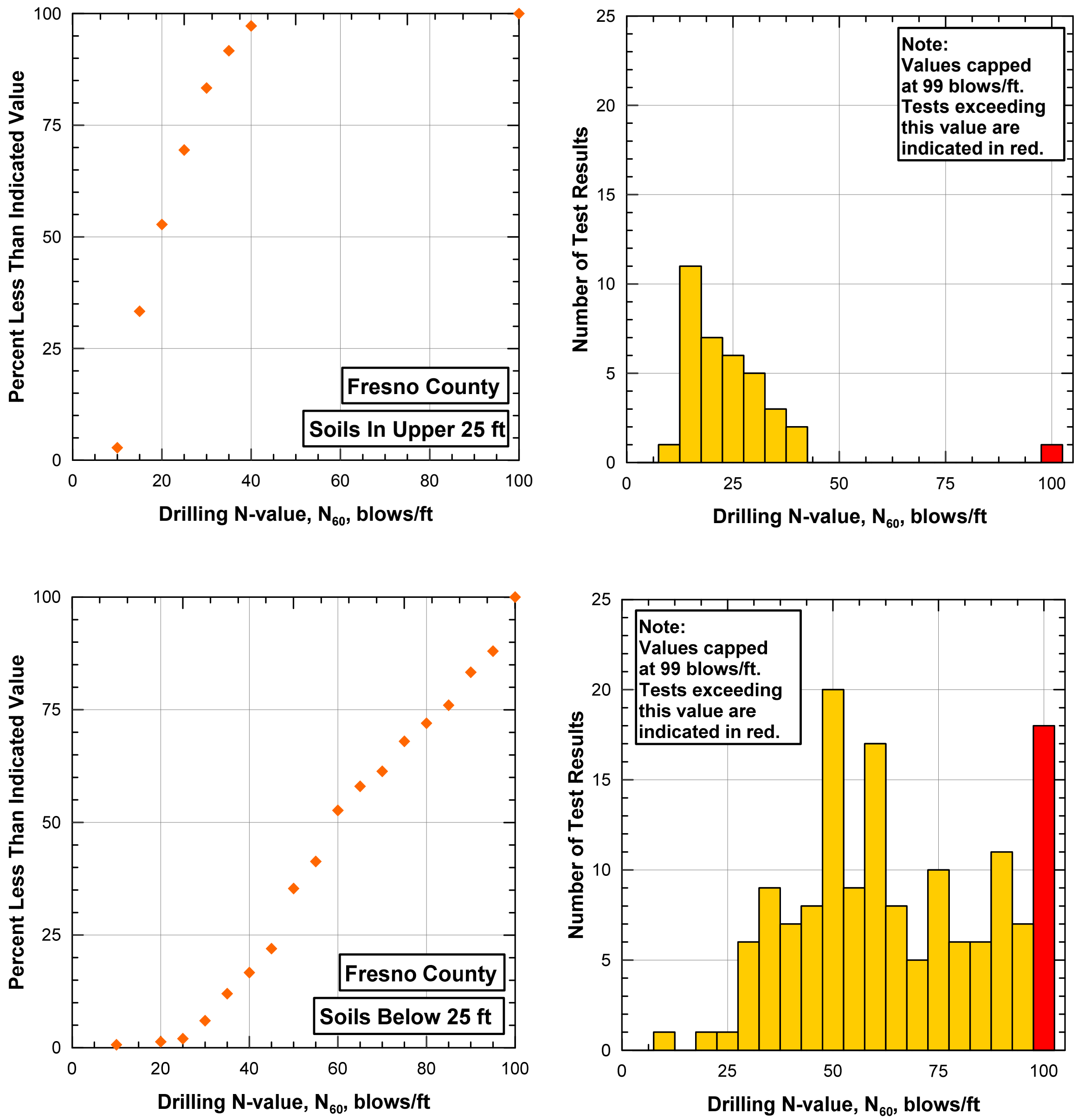


Figure A3.4-2
Statistical Summary of SPT N_{60} – Fresno County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

A3.5 Cone Tip Resistance

Table A3.5-1
Statistical Summary of Cone Tip Resistance – Fresno County

Cone Tip Resistance	CPT	
	Upper 25 ft	Below 25 ft
No. Tests	4981	19656
Mean, tsf	116	197
Median, tsf	93	191
Standard Deviation, tsf	96	89
Maximum, tsf	1000	1000
Minimum, tsf	5.9	5.69

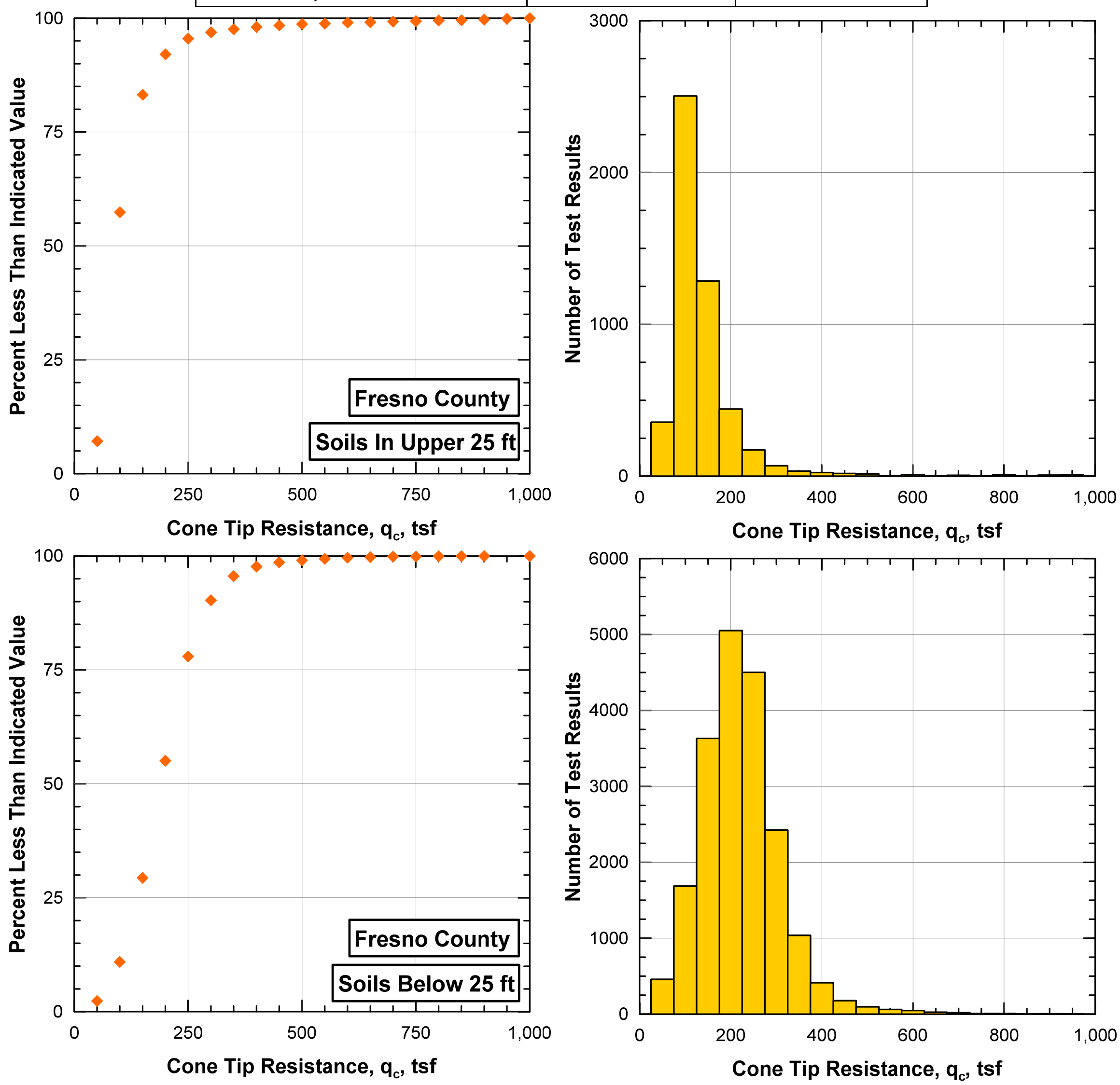


Figure A3.5-1
Statistical Summary of Cone Tip Resistance from CPT Data– Fresno County

A3.6 Soil Modulus

Table A3.6-1
Statistical Summary of Soil Modulus Estimated from SPT Values– Fresno County

Soil Modulus	CPT		Drilling	
	Upper 25 ft	Below 25 ft	Upper 25 ft	Below 25 ft
No. Tests	4554	15926	32	106
Mean, tsf	463	850	197	372
Median, tsf	381	813	168	333
Standard Deviation, tsf	301	311	106	143
Maximum, tsf	2000	2000	700	700
Minimum, tsf	62	99	98	42

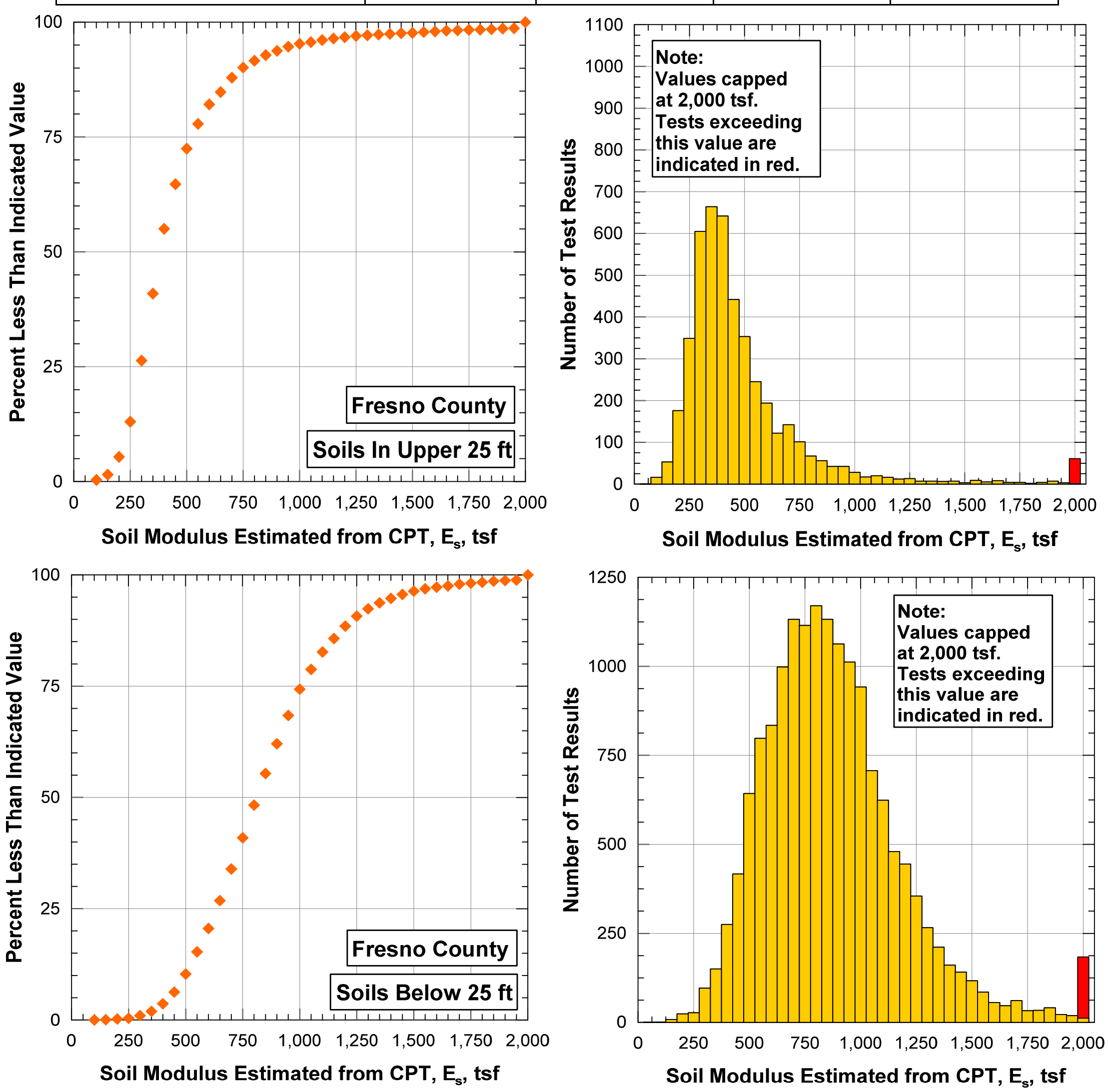


Figure A3.6-1
Statistical Summary of Soil Modulus Estimated from CPT Data– Fresno County

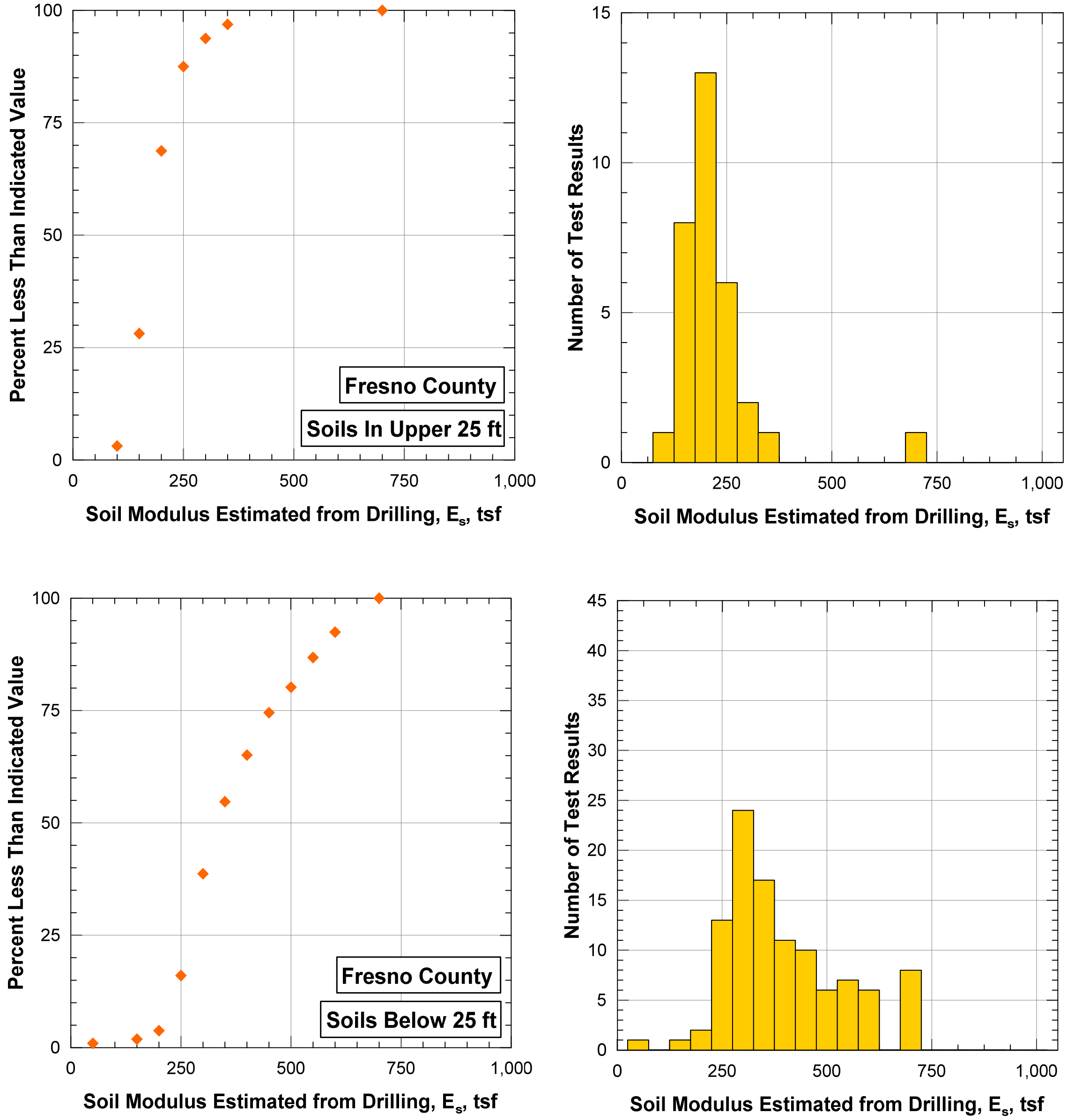


Figure A3.6-2
Statistical Summary of Soil Modulus Estimated from Drilling Data– Fresno County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

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A4.0 Tulare County

The following sections present the results of statistical analyses performed on data obtained from boreholes and CPTs within Tulare County.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in two layers: (1) upper 35 feet of soils (excluding Existing Fill) and (2) soils below 35 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, drilling, or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A4.1 Total Unit Weight

Table A4.1-1
Statistical Summary of Total Unit Weight – Tulare County

Total Unit Weight	CPT				Drilling*			
	Fine		Coarse		Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	762	2441	653	1356	15	18	10	12
Mean, pcf	114.4	116.8	120	122	125	126	120	129
Median, pcf	115.0	115.0	121	124	125	127	121	131
Standard Deviation, pcf	2	5	4	3	4	9	11	5
Maximum, pcf	131	131	127	131	136	135	136	136
Minimum, pcf	111	111	111	115	119	99	100	122

* Unit weight from drilling determined from samplers with full recovery.

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

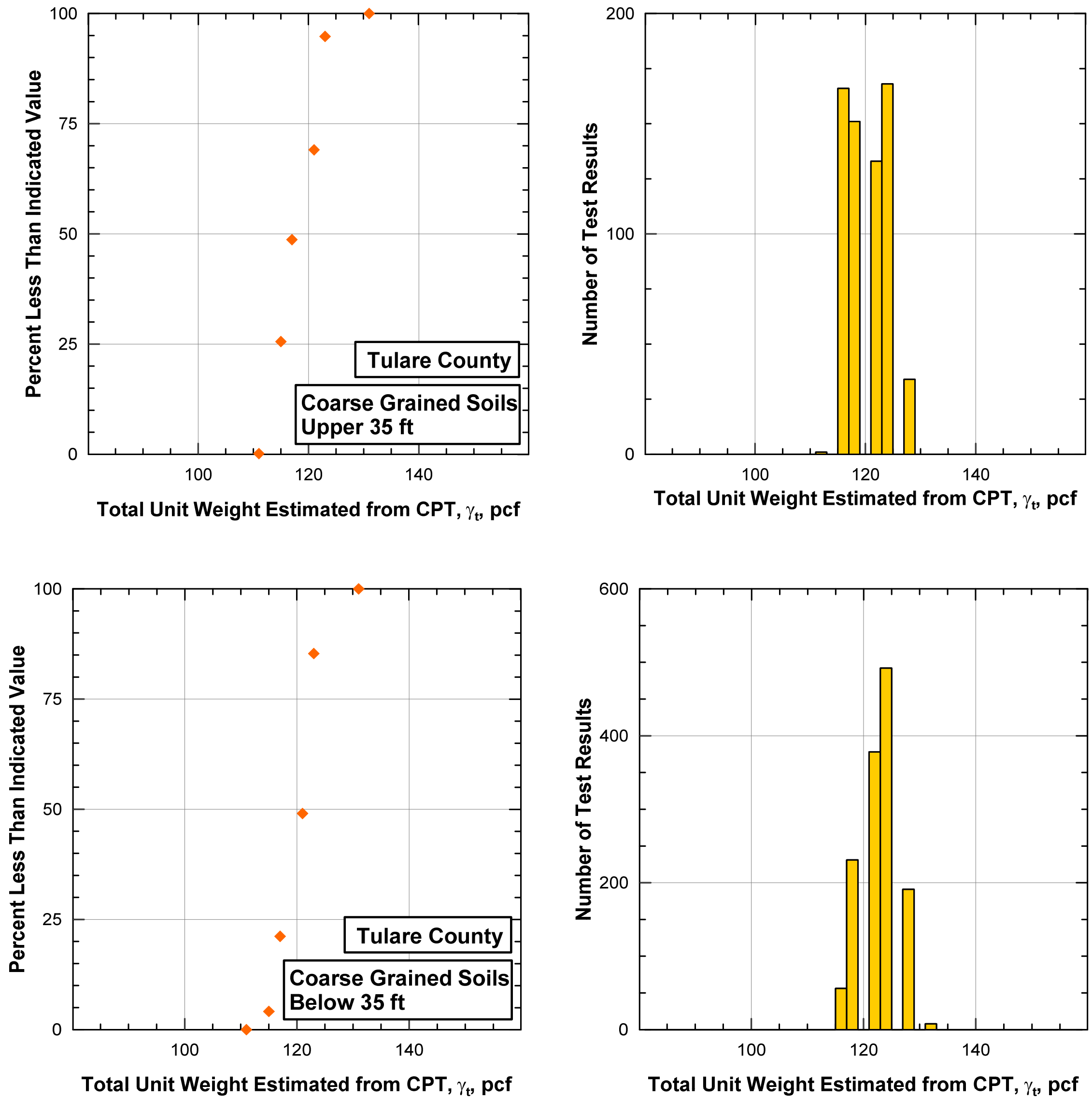

Figure A4.1-1
Statistical Summary of Total Unit Weight Estimated from CPT Data for Coarse Grained Soils – Tulare County

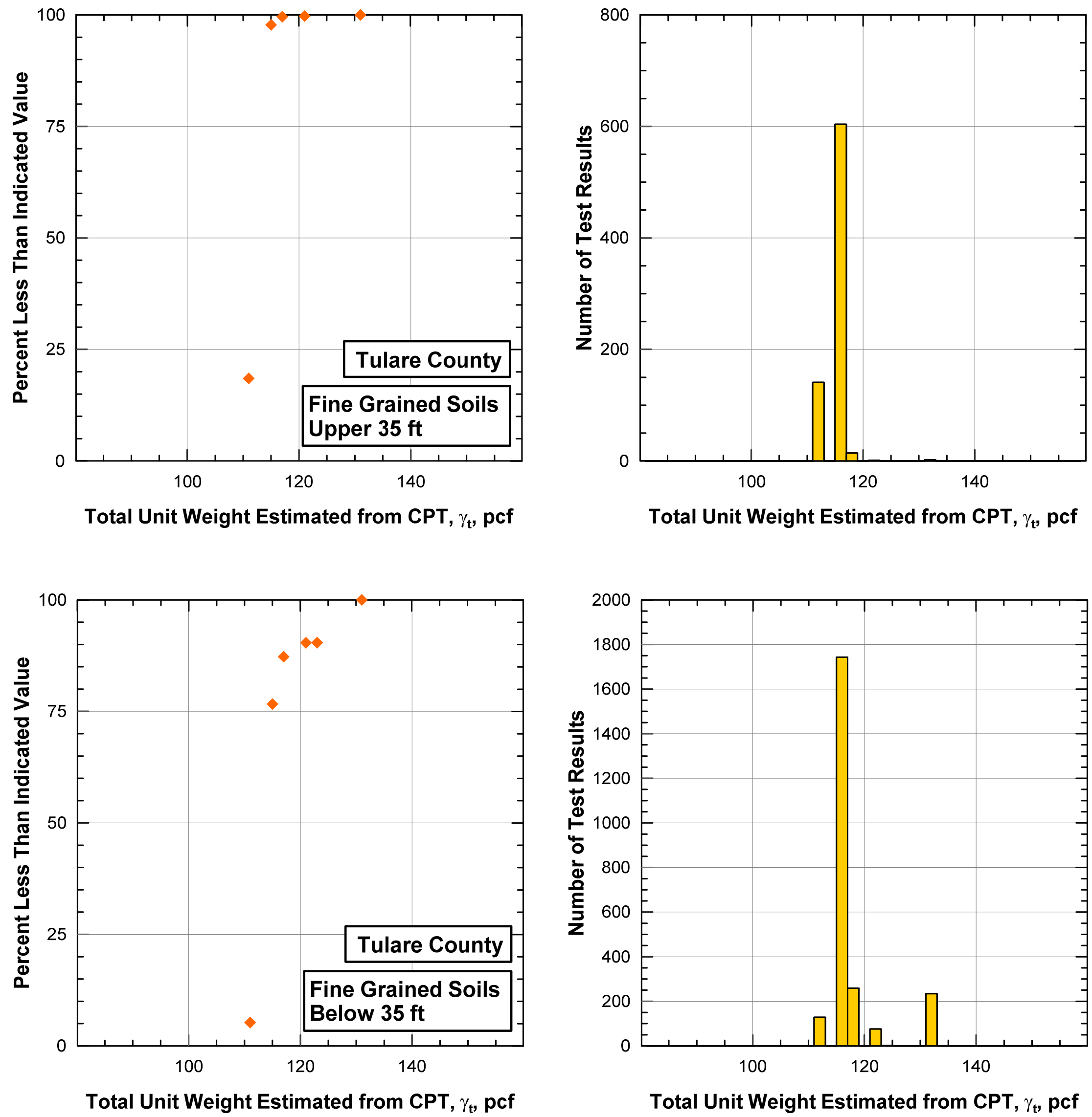

Figure A4.1-2
Statistical Summary of Total Unit Weight Estimated from CPT Data for Fine Grained Soils – Tulare County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

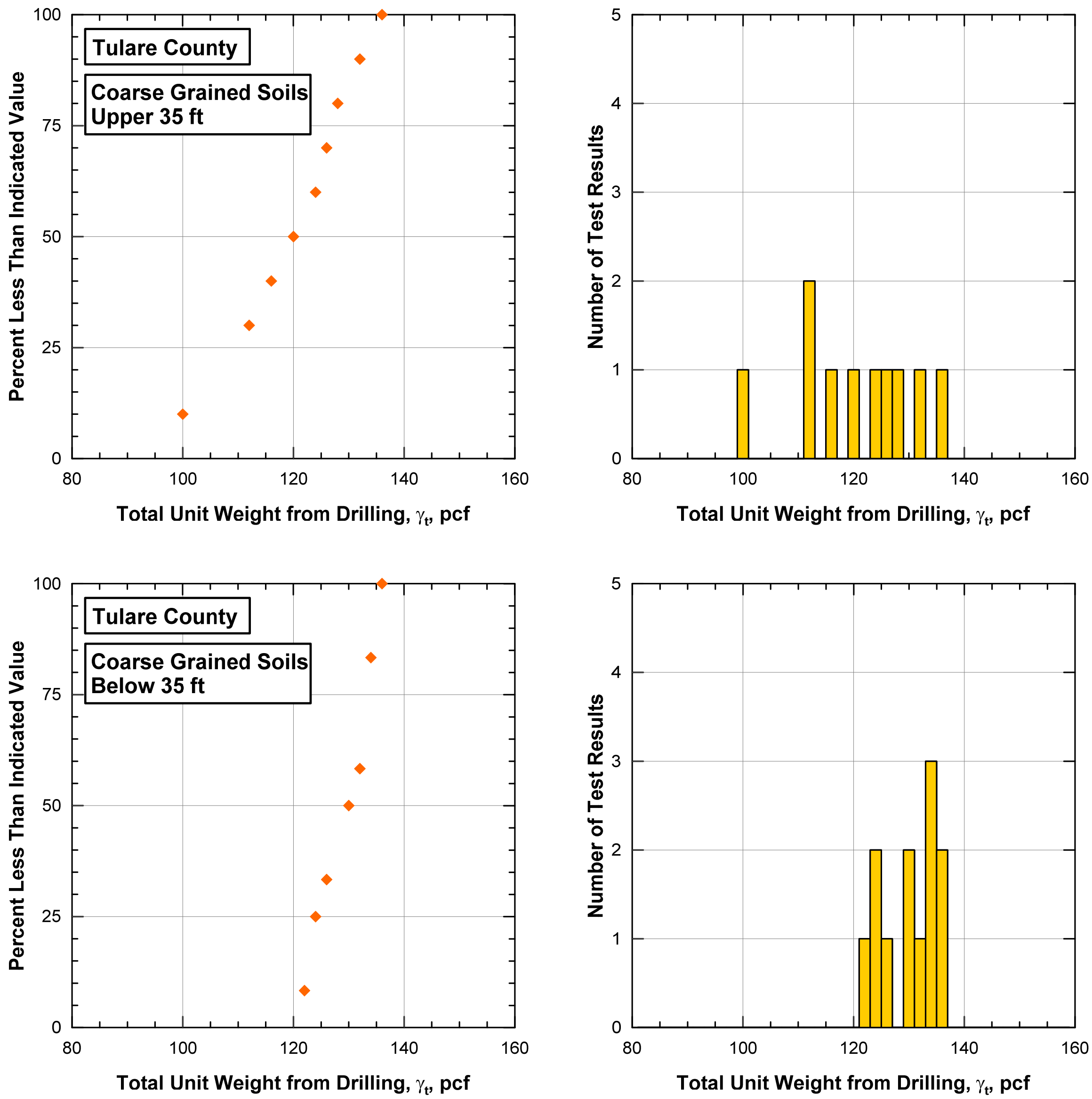

Figure A4.1-3
Statistical Summary of Total Unit Weight from Laboratory Results for Coarse Grained Soils – Tulare County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

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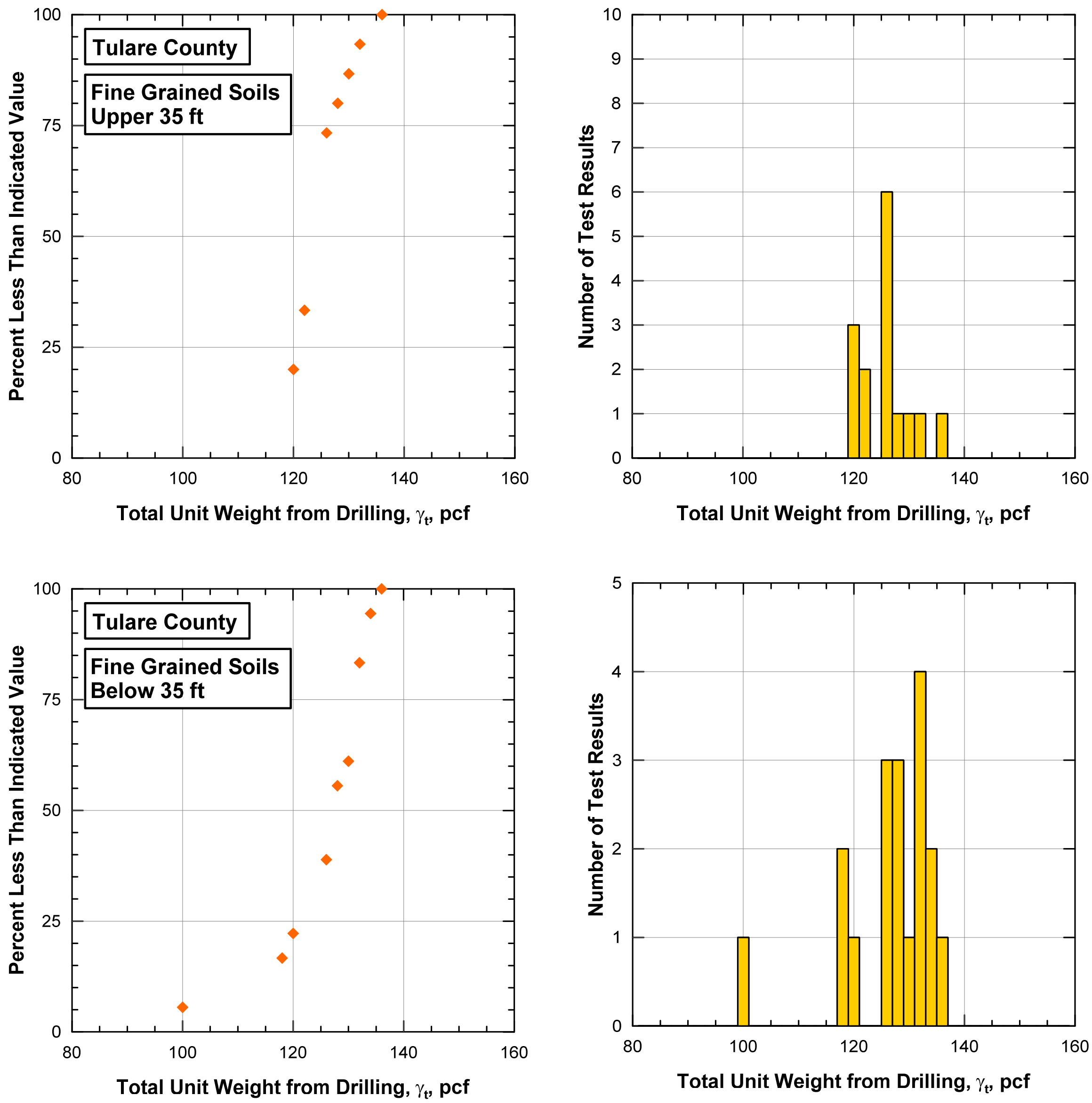

Figure A4.1-4

Statistical Summary of Total Unit Weight from Laboratory Results for Fine Grained Soils – Tulare County

A4.2 Effective Cohesion

Table A4.2-1
Statistical Summary of Effective Cohesion – Tulare County

Effective Cohesion	Laboratory			
	Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	3	2	8	8
Mean, psf	550	890	600	780
Median, psf	400	890	530	900
Standard Deviation, psf	397	156	360	303
Maximum, psf	1000	1000	1000	1000
Minimum, psf	250	780	100	140

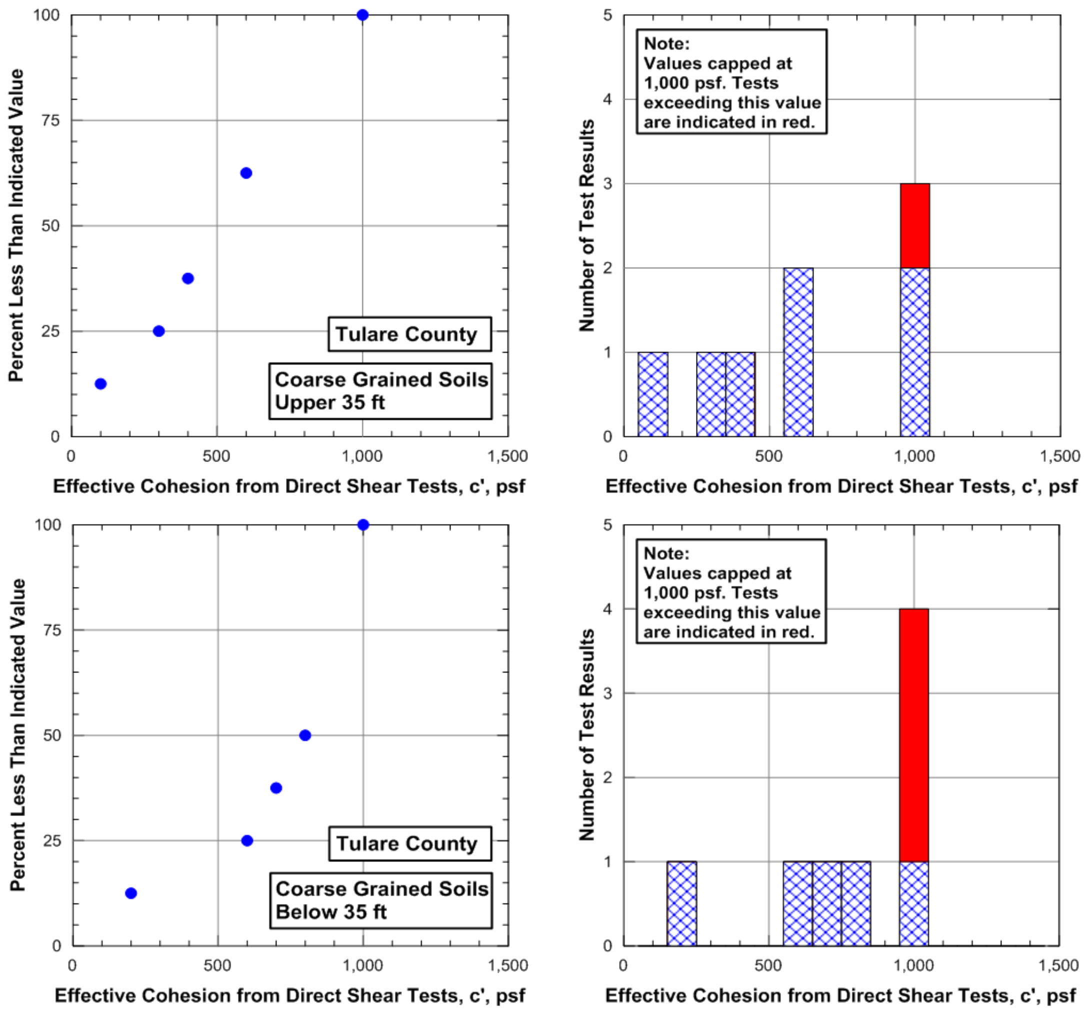

Figure A4.2-1
Statistical Summary of Effective Cohesion from Laboratory Results for Coarse Grained Soils – Tulare County

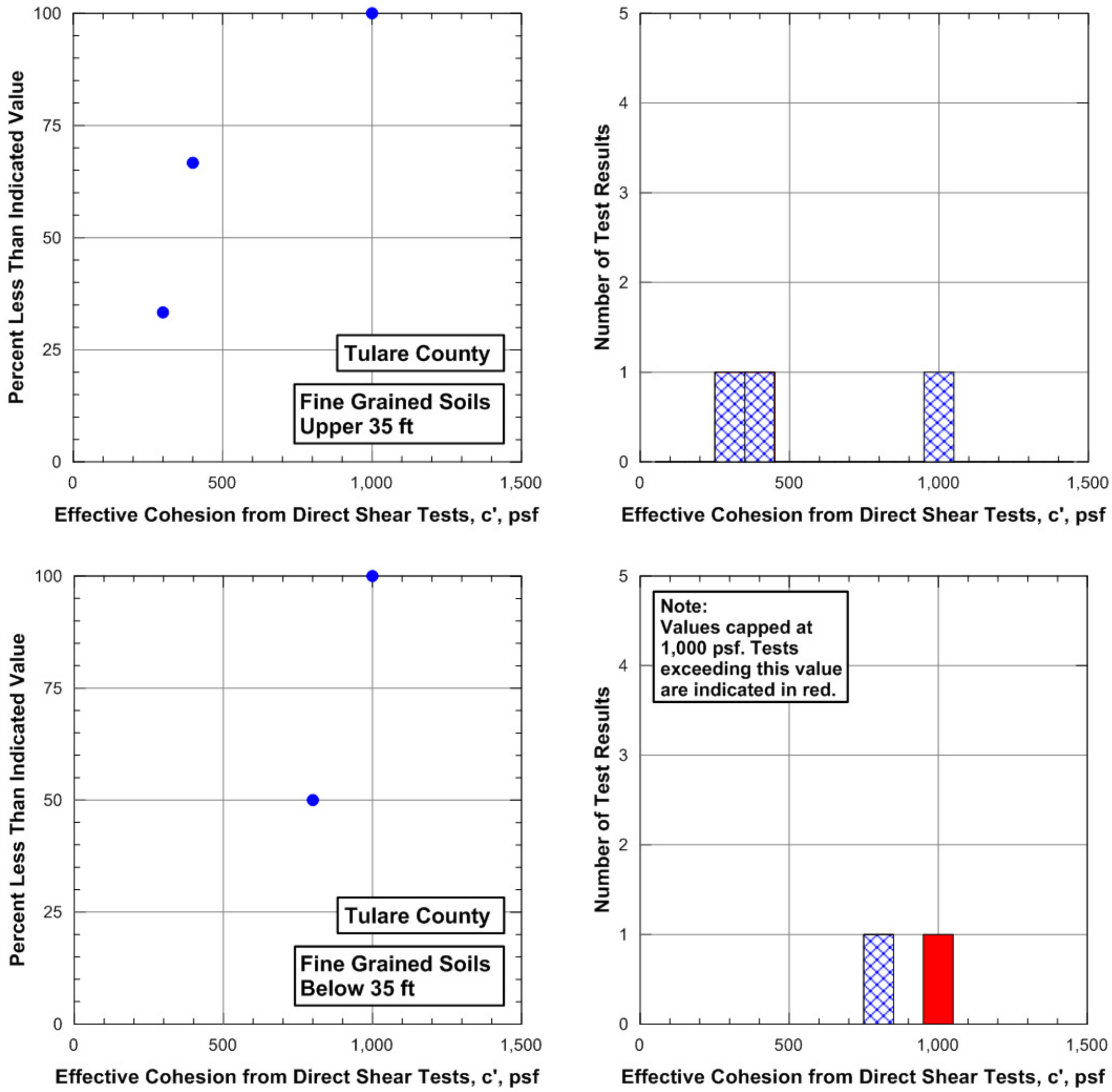

Figure A4.2-2

Statistical Summary of Effective Cohesion from Laboratory Results for Fine Grained Soils – Tulare County

A4.3 Effective Friction Angle

Table A4.3-1
Statistical Summary of Effective Friction Angle for CPT Data – Tulare County

Effective Friction Angle	CPT			
	Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	–	–	2645	5421
Mean, deg	–	–	40	39
Median, deg	–	–	41	39
Standard Deviation, deg	–	–	4	3
Maximum, deg	–	–	50	45
Minimum, deg	–	–	26	25

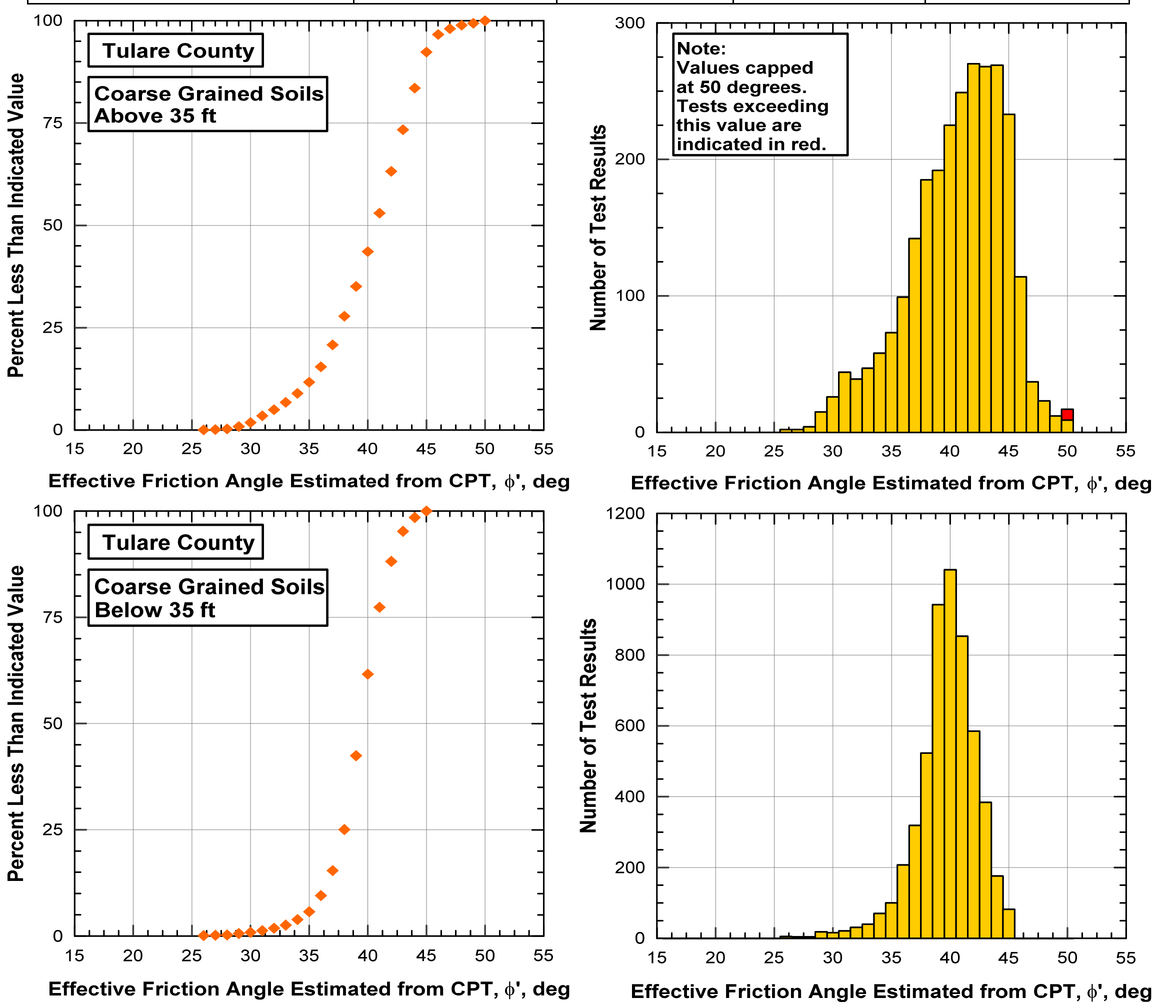

Figure A4.3-1
Statistical Summary of Effective Friction Angle Estimated from CPT Data for Coarse Grained Soils– Tulare County

Table A4.3-2
Statistical Summary of Effective Friction Angle for Drilling Data – Tulare County

Effective Friction Angle	Drilling		Laboratory	
	Coarse		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	29	80	9	8
Mean, deg	42	46	33	31
Median, deg	44	48	35	31
Standard Deviation, deg	7	4	7	5
Maximum, deg	50	50	39	40
Minimum, deg	20	32	20	22

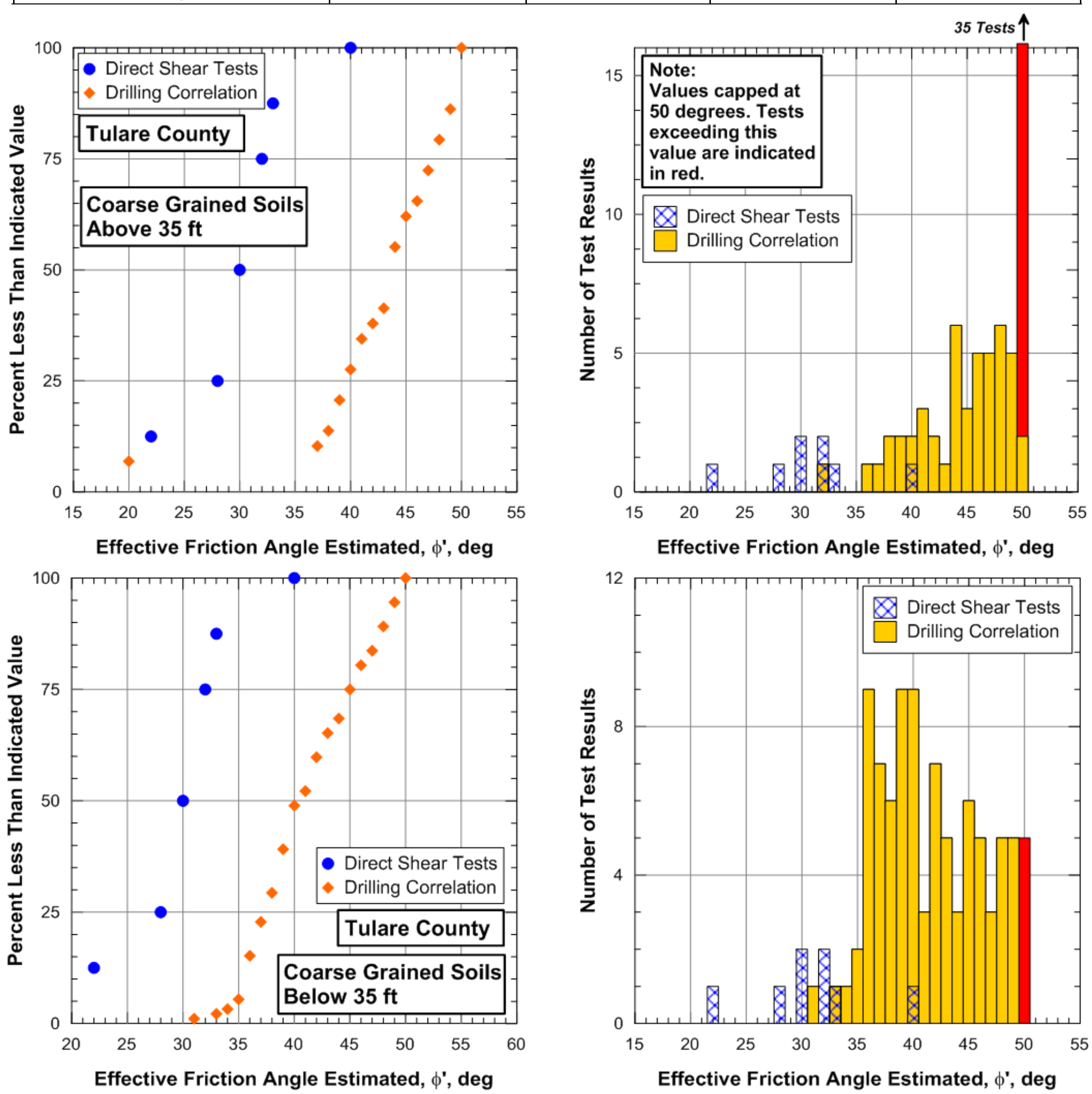

Figure A4.3-2
Statistical Summary of Effective Friction Angle Estimated from Drilling and Laboratory Data for Coarse Grained Soil – Tulare County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

A4.4 SPT N_{60}

Table A4.4-1
Statistical Summary of SPT N60 – Tulare County

SPT N_{60}	CPT				Drilling			
	Fine		Coarse		Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	3082	9787	2645	5421	31	92	25	80
Mean, blows/ft	11	23	26	57	17	36	30	61
Median, blows/ft	10	20	24	56	16	31	31	63
Standard Deviation, blows/ft	5	13	14	19	7	18	12	25
Maximum, blows/ft	33	99	73	99	35	99	64	99
Minimum, blows/ft	2	5	3	6	7	8	12	10

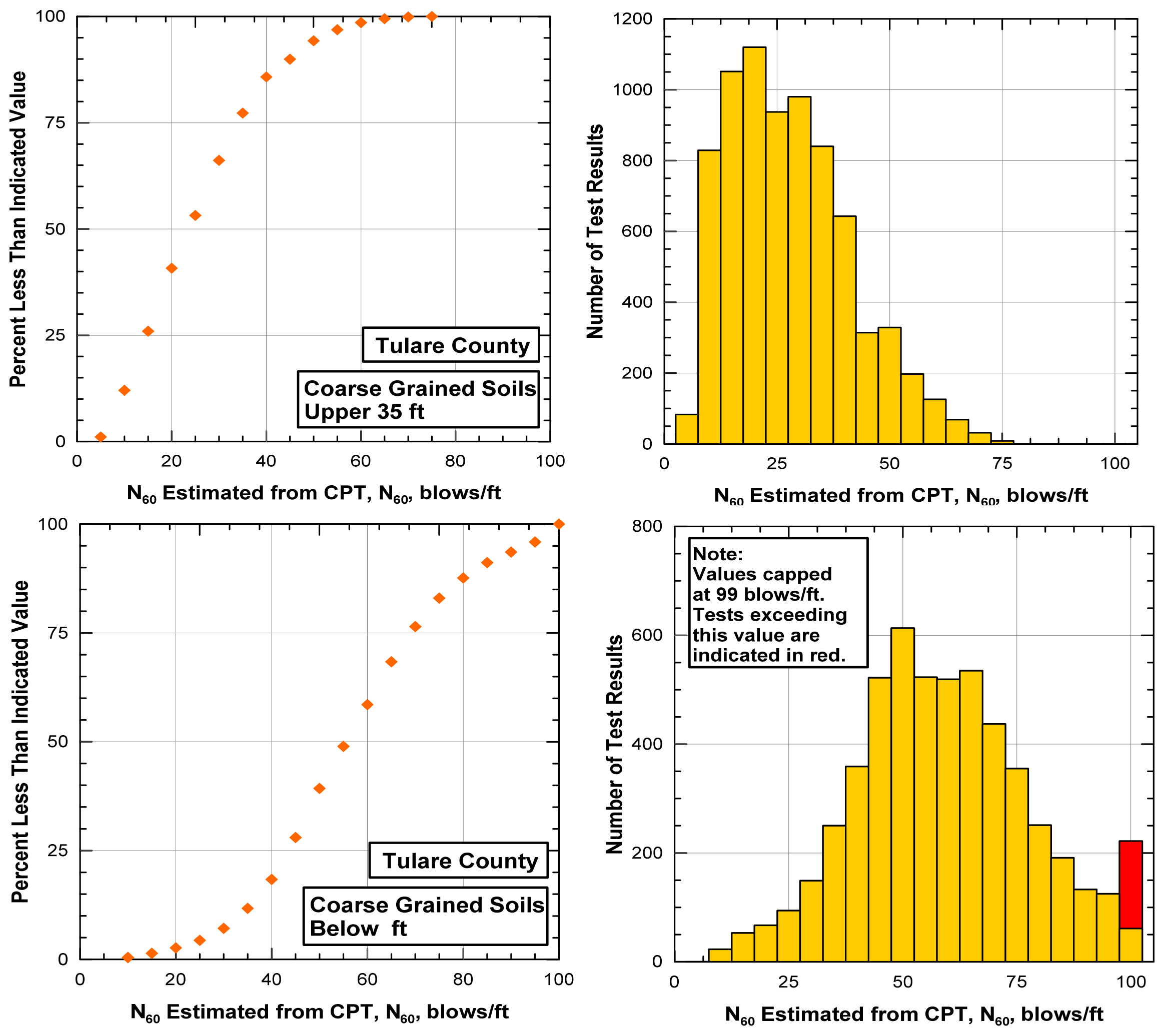

Figure A4.4-1
Statistical Summary of SPT N_{60} Estimated from CPT Data for Coarse Grained Soils – Tulare County

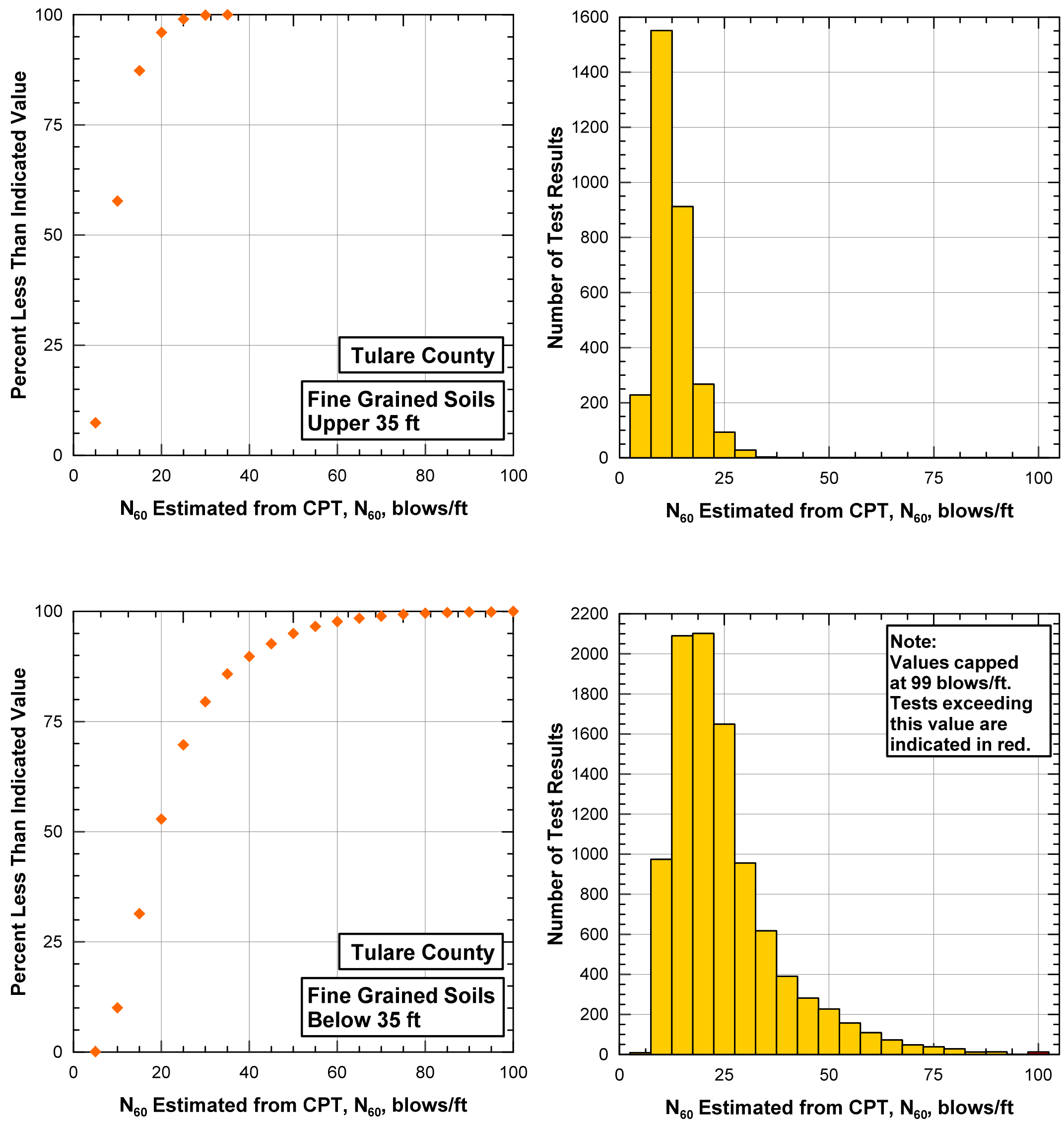

Figure A4.4-2

Statistical Summary of SPT N_{60} Estimated from CPT Data for Fine Grained Soils – Tulare County

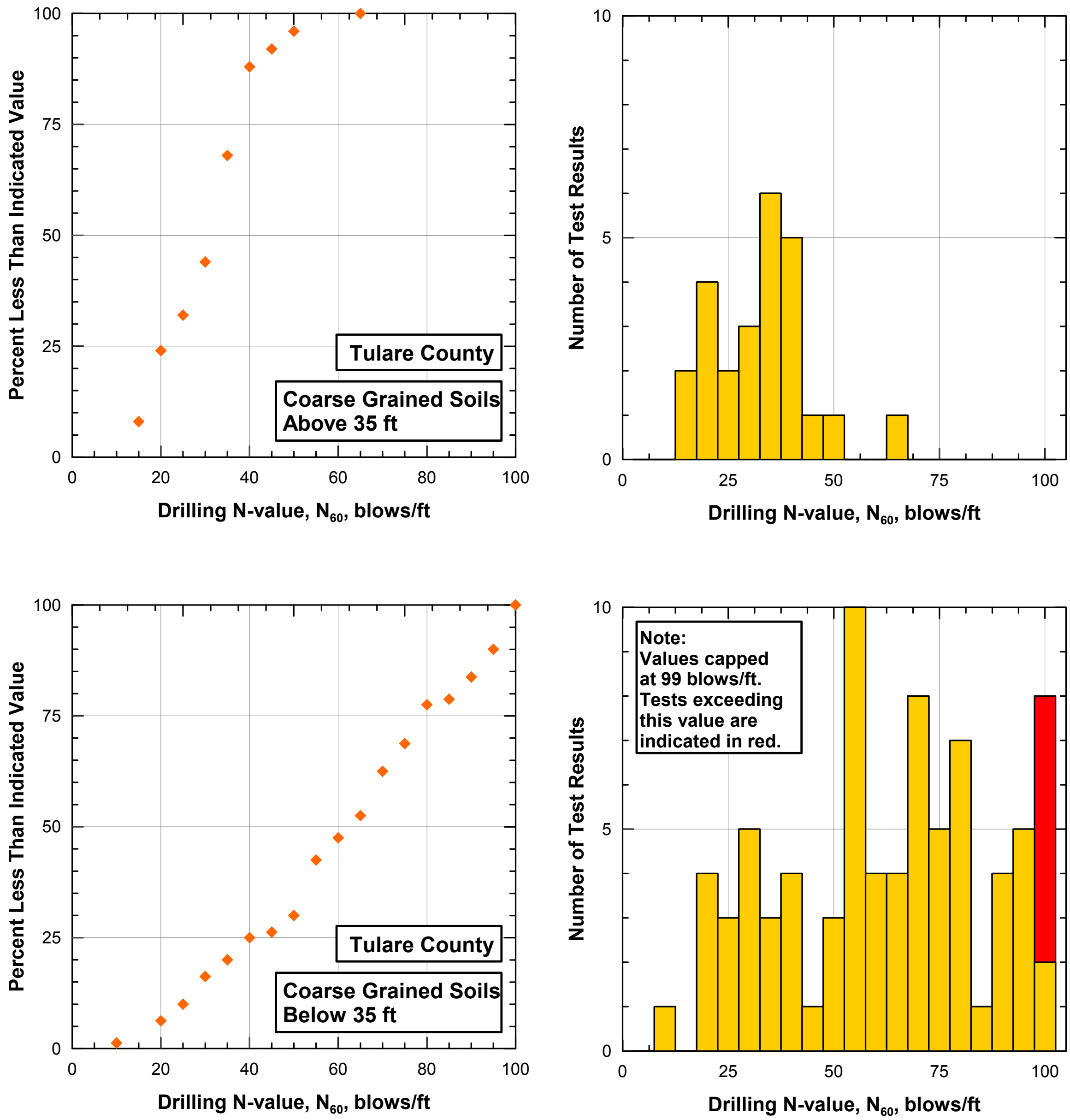

Figure A4.4-3
Statistical Summary of SPT N_{60} for Coarse Grained Soils– Tulare County

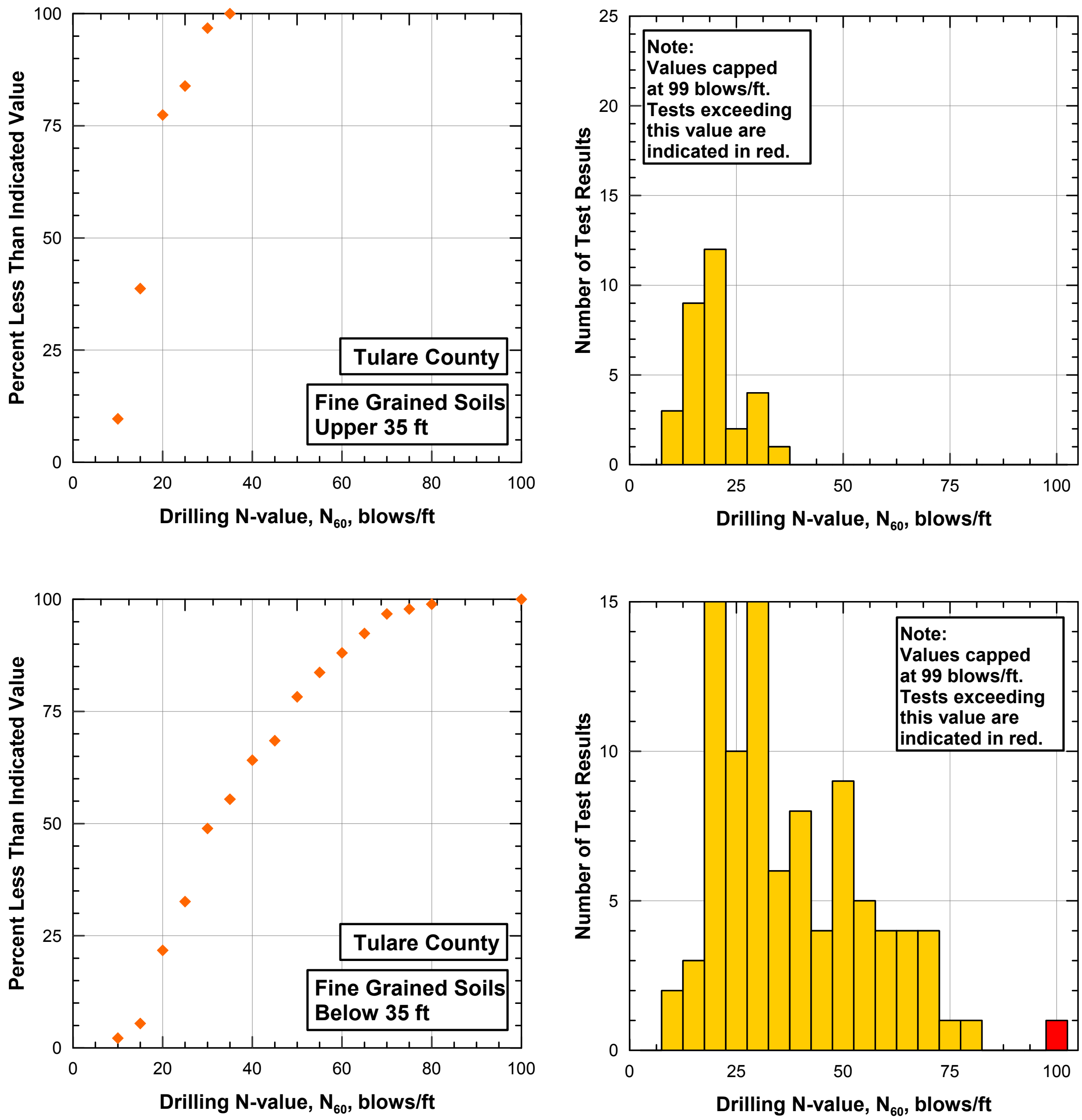

Figure A4.4-4
Statistical Summary of SPT N_{60} for Fine Grained Soils– Tulare County

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RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

A4.5 Cone Tip Resistance

Table A4.5-1
Statistical Summary of Cone Tip Resistance – Tulare County

Cone Tip Resistance	CPT			
	Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	3077	9787	2645	5421
Mean, tsf	26	53	114	238
Median, tsf	23	42	94	237
Standard Deviation, tsf	13	38	80	91
Maximum, tsf	104	365	468	602
Minimum, tsf	5	7	7	16

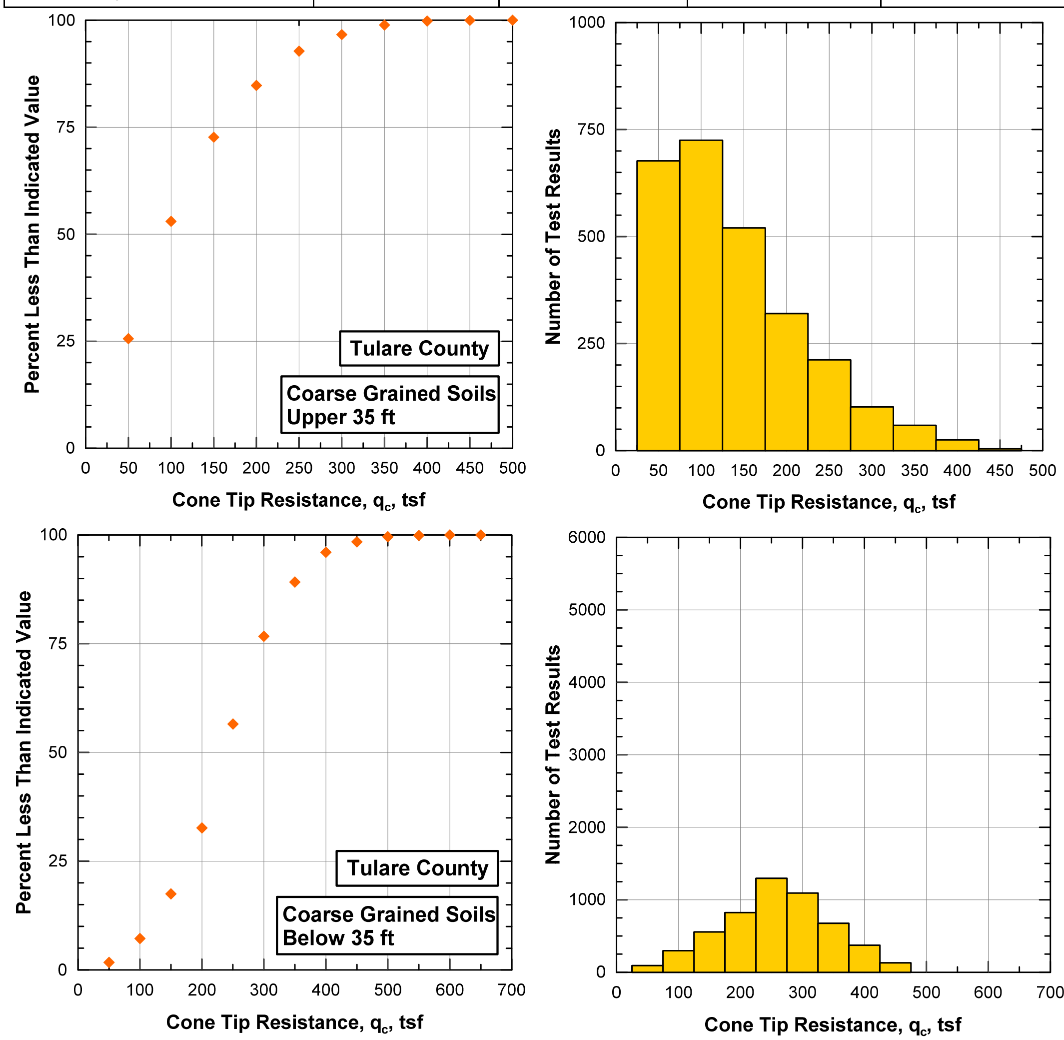

Figure A4.5-1
Statistical Summary of Cone Tip Resistance from CPT Data for Coarse Grained Soils – Tulare County

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RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

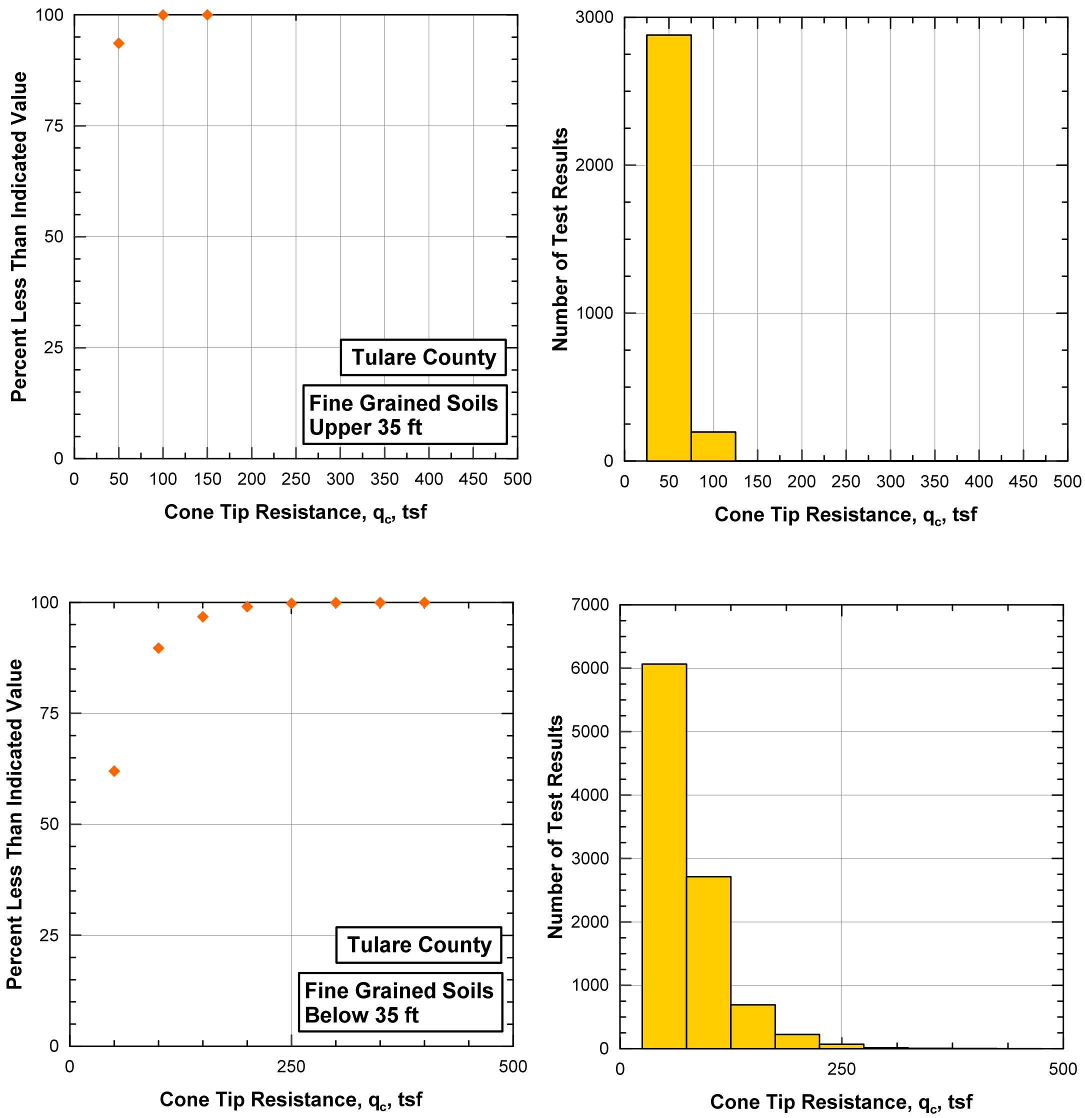

Figure A4.5-2
Statistical Summary of Cone Tip Resistance from CPT Data for Fine Grained Soils – Tulare County

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A4.6 Soil Modulus

Table A4.6-1
Statistical Summary of Soil Modulus Estimated from CPT– Tulare County

Soil Modulus	CPT			
	Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	3082	9787	2645	5421
Mean, tsf	436	804	456	951
Median, tsf	378	652	374	947
Standard Deviation, tsf	237	509	321	364
Maximum, tsf	1802	2000	1871	2000
Minimum, tsf	31	93	30	63

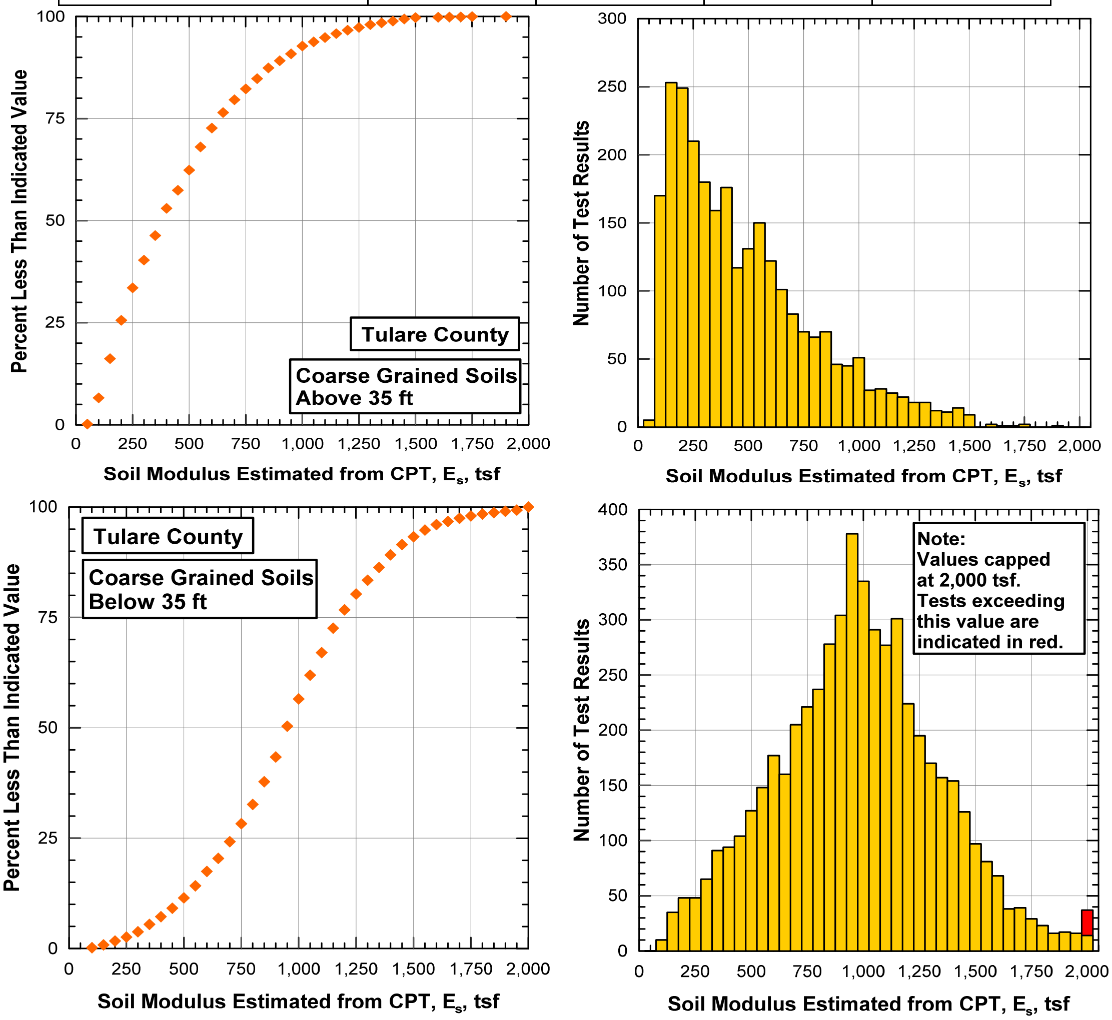

Figure A4.6-1
Statistical Summary of Soil Modulus Estimated from CPT Data for Coarse Grained Soils – Tulare County

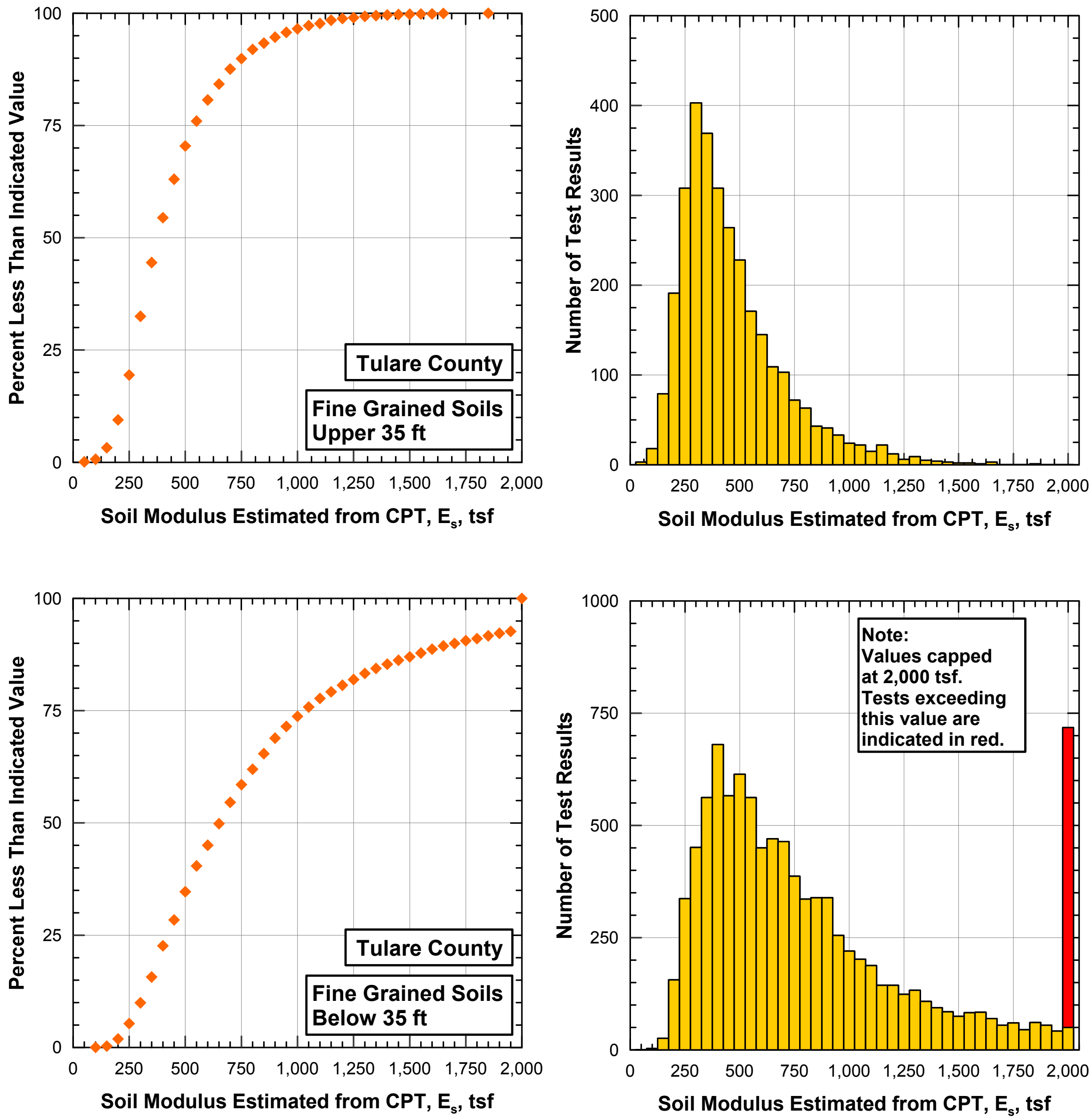

Figure A4.6-2

Statistical Summary of Soil Modulus Estimated from CPT Data for Fine Grained Soils – Tulare County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Table A4.6-2
Statistical Summary of Soil Modulus Estimated from Drilling – Tulare County

Soil Modulus	Drilling			
	Fine		Coarse	
	Upper 35 ft	Below 35 ft	Upper 35 ft	Below 35 ft
No. Tests	-	-	29	80
Mean, tsf	-	-	225	369
Median, tsf	-	-	247	359
Standard Deviation, tsf	-	-	123	146
Maximum, tsf	-	-	462	700
Minimum, tsf	-	-	0	61

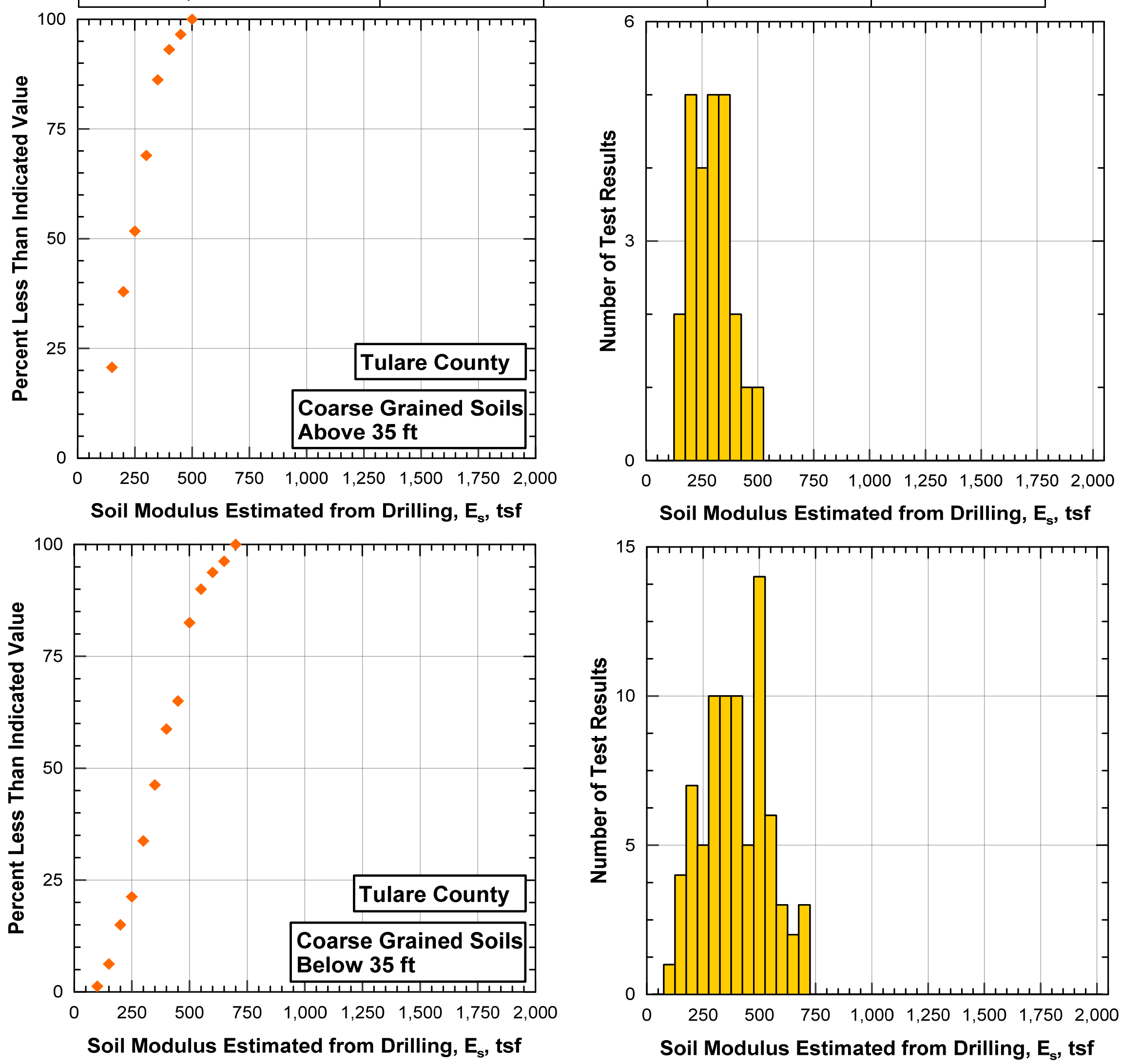

Figure A4.6-3
Statistical Summary of Soil Modulus Estimated from Drilling Data for Coarse Grained Soils – Tulare County

A4.7 Undrained Shear Strength

Table A4.7-1
Statistical Summary of Soil Undrained Shear Strength
from Laboratory Data– Tulare County

Shear Strength	Laboratory	
	Fine	
	Upper 35 ft	Below 35 ft
No. Tests	9	25
Mean, psf	2539	2618
Median, psf	2624	2477
Standard Deviation, psf	533	1212
Maximum, psf	3262	5000
Minimum, psf	1880	1065

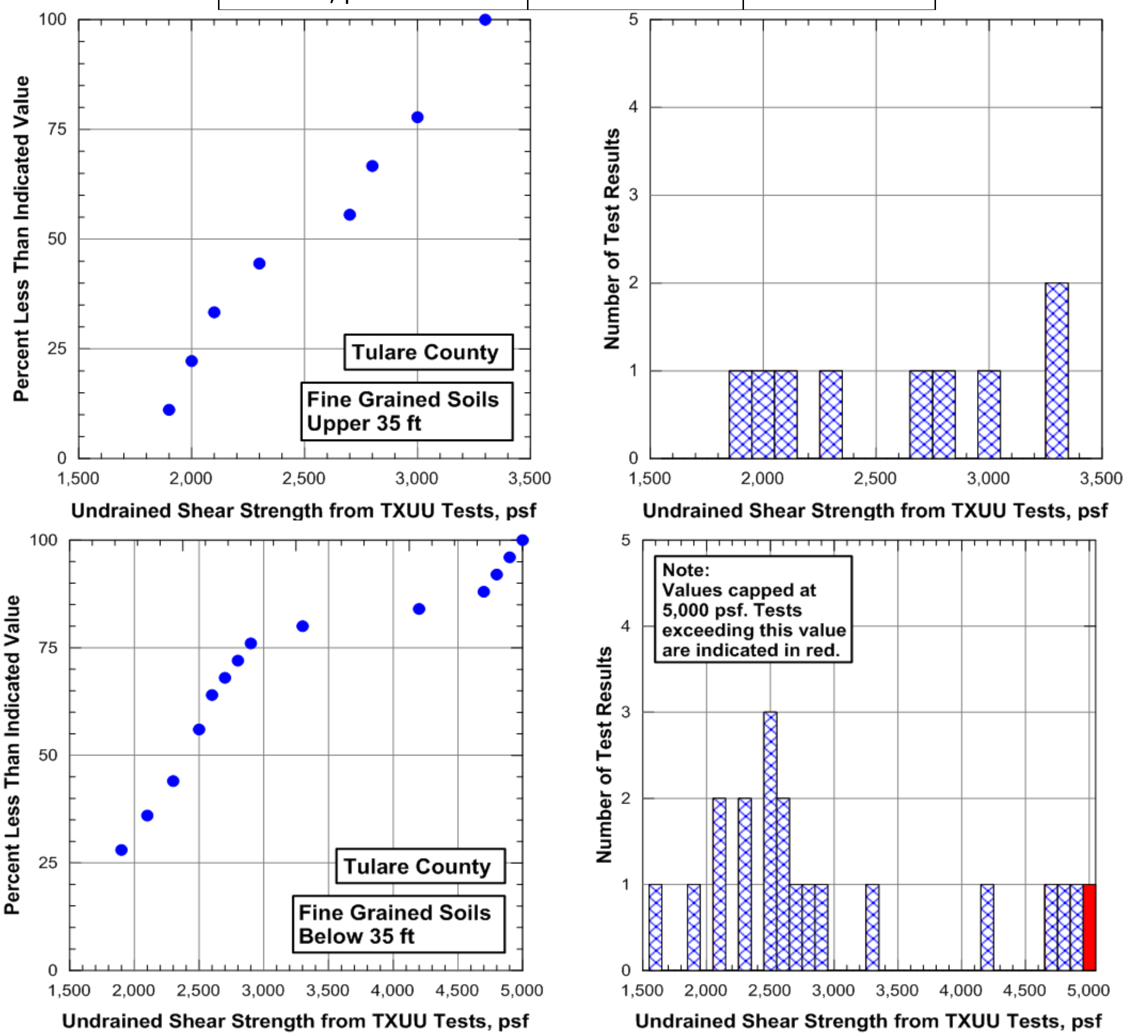

Figure A4.7-1
Statistical Summary of Undrained Shear Strength from Labratory Data for Fine Grained Soils – Tulare County

Table A4.7-2
Statistical Summary of Soil Undrained Shear Strength from CPT Data– Tulare County

Shear Strength	CPT	
	Fine	
	Upper 35 ft	Below 35 ft
No. Tests	3082	9787
Mean, psf	2761	3875
Median, psf	2515	4345
Standard Deviation, psf	1218	1230
Maximum, psf	5000	5000
Minimum, psf	192	614

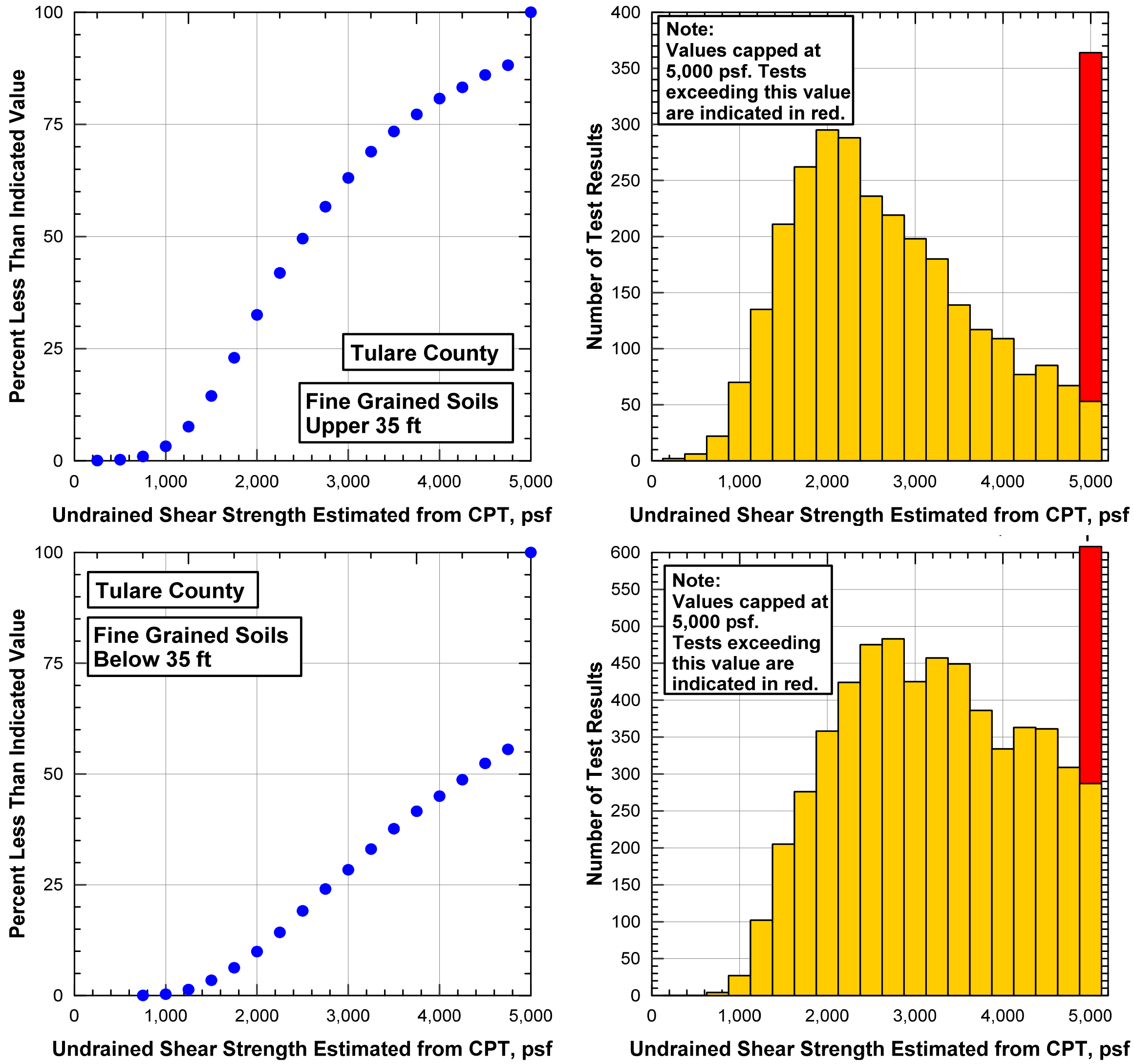

Figure A4.7-2
Statistical Summary of Undrained Shear Strength from CPT Data for Fine Grained Soils – Tulare County

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

A4.8 Assessment of Interbedding of Coarse and Fine Grained Layers in Native Soils of Tulare County

Stratigraphic layers, as identified in the Cone Penetration Test data (CPT) data, were sorted into 'coarse grained' and 'fine grained' layers as per the definitions of Section 6.1.4 of the CP2-3 GBR-B. This section summarizes the methodology employed in the assessment, and presents tabulations of the results.

A4.8.1 Cone Penetration Tests

The CPT data provides a reasonable basis for interpreting coarse and fine grained materials, using the approach cited in the report. This information was assessed using the following procedure:

1. Compute the Normalized Soil Behavior Type (SBT_N) for all the CPT data.
2. Average SBT_N over 1-foot intervals to reduce the data set and discretize layers into a minimum threshold thickness.
3. Categorize each 1-foot layer as either coarse or fine grained.
4. Combine like materials and stack results into columns of layer transitions. A layer was included in the 'Above 35ft' group if the layer immediately above it terminated at less than 34 feet below ground surface. This provided a reasonable filter approximately the 35-foot-depth criteria, without splitting layers. Two notable exceptions are cited, and were manually checked to confirm their effect on results.
5. Compute statistic for each CPT, and for overall data sets, shown in Table A4.8-1. 'COUNT' equals the number of discreet layers for either 'ALL SOILS', 'COARSE', or 'FINE', as noted. Percentages are also computed. The 'SUM' of layer thickness divided by the number of layers (COUNT) yields the average thickness per layer.
6. Inspect results for trends. Columns are color-coded as noted. A consistent increase in percent of fine grained material below 35 feet can be observed, beginning from approximately S0203CPT and continuing southward to the last CPT along the CP2-3 alignment.

The results provide greater resolution upon which to make interpretations, given the frequent readings taken with depth and the larger number of tests in comparison to the boreholes.

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Table A4.8-1
CPT-Based Assessment of Interbedding of Coarse and Fine Grained Layers in Native Soils of Tulare County

CPT ID	Depth Range	LAYERS: ALL SOILS (COARSE & FINE)						LAYERS: COARSE						LAYERS: FINES						PERCENTAGE		% - Full Depth		% - Above 35ft		% - Below 35ft	
		COUNT	SUM	MIN	MAX	MEAN	STDEV	COUNT	SUM	MIN	MAX	MEAN	STDEV	COUNT	SUM	MIN	MAX	MEAN	STDEV	COARSE	FINE	COARSE	FINE	COARSE	FINE	COARSE	FINE
S0186CPT	Full Depth	30	129.5	1.0	12.8	4.3	2.6	15	62.5	1.0	12.8	4.2	2.9	14	61.0	1.0	10.8	4.4	2.4	51%	49%	51%	49%				
	Above 35ft	11	40.4	1.0	6.9	3.7	2.1	5	14.8	1.0	5.9	3.0	2.0	5	19.7	1.0	6.9	3.9	2.2	43%	57%			43%	57%		
	Below 35ft	19	89.1	1.0	12.8	4.7	2.9	10	47.7	1.0	12.8	4.8	3.2	9	41.3	2.0	10.8	4.6	2.6	54%	46%					54%	46%
S0188CPT	Full Depth	26	95.7	0.5	16.7	3.7	4.8	12	52.2	1.0	16.7	4.3	5.7	13	36.9	0.5	15.7	2.8	4.2	59%	41%	59%	41%				
	Above 35ft	11	35.2	1.0	15.7	3.2	4.5	5	22.6	1.0	15.7	4.5	6.4	5	5.9	1.0	2.0	1.2	0.4	79%	21%			79%	21%		
	Below 35ft	15	60.5	0.5	16.7	4.0	5.2	7	29.5	1.0	16.7	4.2	5.6	8	31.0	0.5	15.7	3.9	5.1	49%	51%					49%	51%
S0190CPT	Full Depth	30	149.4	1.0	18.7	5.0	5.0	15	82.2	1.0	17.7	5.5	4.8	14	61.0	1.0	18.7	4.4	5.4	57%	43%	57%	43%				
	Above 35ft	7	34.7	1.0	10.8	5.0	3.7	3	22.6	3.0	10.8	7.5	4.1	3	5.9	1.0	3.0	2.0	1.0	79%	21%			79%	21%		
	Below 35ft	23	114.7	1.0	18.7	5.0	5.4	12	59.5	1.0	17.7	5.0	5.0	11	55.1	1.0	18.7	5.0	6.0	52%	48%					52%	48%
S0191CPT	Full Depth	41	149.5	0.5	15.7	3.6	3.3	20	69.9	1.0	15.7	3.5	3.7	20	69.4	0.5	11.8	3.5	2.7	50%	50%	50%	50%				
	Above 35ft	6	34.9	1.0	11.8	5.8	4.4	3	9.8	1.0	5.9	3.3	2.5	2	14.8	3.0	11.8	7.4	6.3	40%	60%			40%	60%		
	Below 35ft	35	114.7	0.5	15.7	3.3	3.0	17	60.0	1.0	15.7	3.5	3.9	18	54.6	0.5	8.9	3.0	1.9	52%	48%					52%	48%
S0192CPT	Full Depth	27	156.7	0.6	17.7	5.8	4.5	13	85.5	1.0	17.7	6.6	4.7	3	4.6	0.6	14.8	5.0	4.6	57%	43%	57%	43%				
	Above 35ft	8	34.2	0.6	11.8	4.3	3.9	3	7.3	1.0	5.3	2.4	2.5	4	20.3	0.6	11.8	5.1	5.0	26%	74%			26%	74%		
	Below 35ft	19	122.5	1.0	17.7	6.4	4.7	10	78.2	3.4	17.7	7.8	4.5	9	44.3	1.0	14.8	4.9	4.7	64%	36%					64%	36%
S0193CPT	Full Depth	14	99.8	1.0	51.2	7.1	13.0	7	30.5	1.0	10.3	4.4	3.3	6	62.5	1.0	51.2	10.4	20.0	33%	67%	33%	67%				
	Above 35ft	5	75.7	1.5	51.2	15.1	20.4	2	16.2	5.9	10.3	8.1	3.1	2	52.7	1.5	51.2	26.3	35.1	24%	76%			24%	76%		
	Below 35ft	9	24.1	1.0	5.9	2.7	1.7	5	14.3	1.0	5.9	2.9	1.9	4	9.8	1.0	3.9	2.5	1.7	59%	41%					59%	41%
S0194CPT	Full Depth	28	115.6	0.7	19.7	4.1	4.9	14	76.5	1.0	19.7	5.5	5.9	13	32.2	0.7	12.8	2.5	3.2	70%	30%	70%	30%				
	Above 35ft	11	35.4	1.0	9.8	3.2	3.3	5	20.7	1.0	9.8	4.1	4.4	5	7.9	1.0	3.0	1.6	0.9	72%	28%			72%	28%		
	Below 35ft	17	80.2	0.7	19.7	4.7	5.7	9	55.9	1.0	19.7	6.2	6.7	8	24.4	0.7	12.8	3.0	4.0	70%	30%					70%	30%
S0195CPT	Full Depth	18	99.3	1.0	28.5	5.5	7.1	9	34.0	1.0	11.8	3.8	3.6	8	60.0	1.0	28.5	7.5	9.9	36%	64%	36%	64%				
	Above 35ft	5	40.8	1.0	28.5	8.2	11.5	2	4.9	1.0	3.9	2.5	2.1	2	30.5	2.0	28.5	15.3	18.8	14%	86%			14%	86%		
	Below 35ft	13	58.6	1.0	16.7	4.5	4.8	7	29.0	1.0	11.8	4.1	3.9	6	29.5	1.0	16.7	4.9	6.0	50%	50%					50%	50%
S0198CPT	Full Depth	22	89.8	1.0	19.7	4.1	5.4	11	34.0	1.0	8.9	3.1	2.8	10	50.2	1.0	19.7	5.0	7.5	40%	60%	40%	60%				
	Above 35ft	10	34.2	1.0	18.7	3.4	5.6	5	6.9	1.0	2.0	1.4	0.5	4	21.7	1.0	18.7	5.4	8.9	24%	76%			24%	76%		
	Below 35ft	12	55.6	1.0	19.7	4.6	5.4	6	27.1	1.0	8.9	4.5	3.2	6	28.5	1.0	19.7	4.8	7.4	49%	51%					49%	51%
S0199CPT	Full Depth	14	89.8	1.0	17.7	6.4	4.3	7	41.8	1.0	9.8	6.0	2.8	6	42.3	1.0	17.7	7.1	6.1	50%	50%	50%	50%				
	Above 35ft	4	38.1	4.9	17.7	9.5	5.9	2	14.8	4.9	9.8	7.4	3.5	1	17.7	17.7	17.7	17.7	NA	45%	55%			45%	55%		
	Below 35ft	10	51.7	1.0	8.9	5.2	3.0	5	27.1	1.0	8.4	5.4	2.8	5	24.6	1.0	8.9	4.9	3.5	52%	48%					52%	48%
S0200CPT	Full Depth	23	109.8	1.0	17.7	4.8	4.9	11	45.3	1.0	17.7	4.1	5.2	11	59.5	1.0	15.7	5.4	5.0	43%	57%	43%	57%				
	Above 35ft	12	35.5	1.0	7.9	3.0	2.5	6	10.8	1.0	3.9	1.8	1.2	5	19.7	1.0	7.9	3.9	3.3	35%	65%			35%	65%		
	Below 35ft	11	74.3	1.0	17.7	6.8	6.2	5	34.4	1.0	17.7	6.9	7.0	6	39.9	1.0	15.7	6.6	6.1	46%	54%					46%	54%
S0201CPT	Full Depth	19	99.8	1.0	12.8	5.3	3.7	9	31.0	1.0	9.3	3.4	2.8	9	63.0	2.0	12.8	7.0	4.0	33%	67%	33%	67%				
	Above 35ft	6	34.4	1.0	12.8	5.7	4.2	2	6.9	1.0	5.9	3.4	3.5	3	21.7	2.0	12.8	7.2	5.4	24%	76%			24%	76%		
	Below 35ft	13	65.5	1.0	11.8	5.0	3.6	7	24.1	1.0	9.3	3.4	2.9	6	41.3	3.0	11.8	6.9	3.6	37%	63%					37%	63%
S0202CPT	Full Depth	24	99.0	0.5	14.8	4.1	3.5	12	50.7	0. 5	7.9	4.2	2.8	11	43.3	1.0	14.8	3.9	4.4	54%	46%	54%	46%				
	Above 35ft	11	35.5	1.0	6.9	3.2	2.1	5	19.7	1.0	6.9	3.9	2.4	5	10.8	1.0	4.9	2.2	1.6	65%	35%			65%	35%		
	Below 35ft	13	63.5	0.5	14.8	4.9	4.3	7	31.0	0.5	7.9	4.4	3.3	6	32.5	1.0	14.8	5.4	5.6	49%	51%					49%	51%
S0203CPT	Full Depth	23	99.2	1.0	13.8	4.3	3.6	11	38.9	1.0	12.8	3.5	3.6	11	54.1	1.0	13.8	4.9	3.8	42%	58%	42%	58%				
	Above 35ft	8	36.7	1.0	12.8	4.6	3.8	3	17.7	1.0	12.8	5.9	6.1	4	12.8	1.0	4.9	3.2	1.7	58%	42%			58%	42%		
	Below 35ft	15	62.5	1.0	13.8	4.2	3.7	8	21.2	1.0	6.9	2.6	2.1	7	41.3	1.0	13.8	5.9	4.5	34%	66%					34%	66%

RFP No.: HSR 13-57 – Addendum No. 1 - 06/10/2014

Table A4.8-1
CPT-Based Assessment of Interbedding of Coarse and Fine Grained Layers in Native Soils of Tulare County

CPT ID	Depth Range	LAYERS: ALL SOILS (COARSE & FINE)						LAYERS: COARSE						LAYERS: FINES						PERCENTAGE		% - Full Depth		% - Above 35ft		% - Below 35ft	
		COUNT	SUM	MIN	MAX	MEAN	STDEV	COUNT	SUM	MIN	MAX	MEAN	STDEV	COUNT	SUM	MIN	MAX	MEAN	STDEV	COARSE	FINE	COARSE	FINE	COARSE	FINE	COARSE	FINE
S0204CPT	Full Depth	19	102.3	1.0	19.7	5.4	5.2	9	51.2	1.0	16.7	5.7	5.0	9	45.8	1.0	19.7	5.1	5.9	53%	47%	53%	47%				
	Above 35ft	5	34.9	1.0	16.7	7.0	6.5	2	26.6	9.8	16.7	13.3	4.9	2	3.0	1.0	2.0	1.5	0.7	90%	10%			90%	10%		
	Below 35ft	14	67.4	1.0	19.7	4.8	4.8	7	24.6	1.0	6.9	3.5	2.3	7	42.8	1.0	19.7	6.1	6.4	36%	64%					36%	64%
S0206CPT	Full Depth	16	84.2	1.0	14.8	5.3	4.6	8	19.2	1.0	6.9	2.4	2.1	7	59.1	1.0	14.8	8.4	5.0	25%	75%	25%	75%				
	Above 35ft	8	40.4	1.0	13.8	5.1	4.5	4	11.8	1.0	6.9	3.0	2.8	3	22.6	1.0	13.8	7.5	6.4	34%	66%			34%	66%		
	Below 35ft	8	43.8	1.0	14.8	5.5	5.0	4	7.4	1.0	3.9	1.8	1.4	4	36.4	4.9	14.8	9.1	4.6	17%	83%					17%	83%
S0208CPT	Full Depth	25	102.4	1.0	9.8	4.1	3.1	12	39.4	1.0	7.9	3.3	2.4	12	56.6	1.0	9.8	4.7	3.8	41%	59%	41%	59%				
	Above 35ft	10	37.0	1.0	9.8	3.7	3.5	5	17.7	1.0	7.9	3.5	3.2	4	12.8	1.0	9.8	3.2	4.4	58%	42%			58%	42%		
	Below 35ft	15	65.5	1.0	9.8	4.4	3.0	7	21.7	1.0	5.9	3.1	1.9	8	43.8	1.0	9.8	5.5	3.4	33%	67%					33%	67%
S0210CPT	Full Depth	24	111.5	1.0	20.7	4.6	4.9	12	32.0	1.0	4.9	2.7	1.1	11	72.8	1.0	20.7	6.6	6.7	31%	69%	31%	69%				
	Above 35ft	13	49.0	1.0	15.7	3.8	4.0	6	17.7	1.0	4.9	3.0	1.4	6	24.6	1.0	15.7	4.1	5.8	42%	58%			42%	58%		
	Below 35ft	11	62.5	1.0	20.7	5.7	5.9	6	14.3	2.0	3.4	2.4	0.7	5	48.2	1.0	20.7	9.6	7.1	23%	77%					23%	77%
S0211CPT	Full Depth	28	122.8	0.5	13.8	4.4	3.6	14	50.7	0.5	12.8	3.6	3.6	13	65.9	1.0	13.8	5.1	3.8	43%	57%	43%	57%				
	Above 35ft	11	34.7	1.0	6.9	3.2	2.3	5	14.8	1.0	6.9	3.0	2.4	5	13.8	1.0	5.9	2.8	2.1	52%	48%			52%	48%		
	Below 35ft	17	88.1	0.5	13.8	5.2	4.2	9	35.9	0.5	12.8	4.0	4.2	8	52.2	2.0	13.8	6.5	3.9	41%	59%					41%	59%
S0212CPT	Full Depth	19	143.9	1.0	56.1	7.6	13.0	9	23.6	1.0	7.9	2.6	2.7	9	114.7	1.0	56.1	12.7	17.7	17%	83%	17%	83%				
	Above 35ft	11	40.1	1.0	16.7	3.6	4.8	5	11.8	1.0	6.9	2.4	2.6	5	22.6	1.0	16.7	4.5	6.8	34%	66%			34%	66%		
	Below 35ft	8	103.8	1.0	56.1	13.0	18.5	4	11.8	1.0	7.9	3.0	3.3	4	92.0	4.9	56.1	23.0	22.8	11%	89%					11%	89%
S0214CPT	Full Depth	16	142.1	0.5	41.3	8.9	11.7	8	24.1	0.5	8.9	3.0	2.9	7	112.2	1.0	41.3	16.0	15.1	18%	82%	18%	82%				
	Above 35ft	7	60.0	1.0	30.5	8.6	10.3	3	12.8	2.0	8.9	4.3	4.0	3	41.3	1.0	30.5	13.8	15.2	24%	76%			24%	76%		
	Below 35ft	9	82.2	0.5	41.3	9.1	13.4	5	11.3	0.5	5.9	2.3	2.1	4	70.9	1.0	41.3	17.7	17.2	14%	86%					14%	86%
S0216CPT	Full Depth	26	132.6	1.0	25.1	5.1	6.3	12	34.4	1.0	9.8	2.9	2.6	13	93.0	1.0	25.1	7.2	8.2	27%	73%	27%	73%				
	Above 35ft	10	37.6	1.0	9.8	3.8	3.0	4	15.7	1.0	9.8	3.9	4.0	5	16.7	1.0	7.9	3.3	2.7	48%	52%			48%	52%		
	Below 35ft	16	95.0	1.0	25.1	5.9	7.7	8	18.7	1.0	5.9	2.3	1.7	8	76.3	2.0	25.1	9.5	9.7	20%	80%					20%	80%
S0218CPT	Full Depth	11	99.5	1.0	47.7	9.0	14.1	5	14.8	1.0	9.8	3.0	3.9	5	78.2	2.0	47.7	15.6	19.5	16%	84%	16%	84%				
	Above 35ft	7	39.0	1.0	20.7	5.6	7.0	3	3.9	1.0	2.0	1.3	0.6	3	28.5	2.0	20.7	9.5	9.9	12%	88%			12%	88%		
	Below 35ft	4	60.5	1.0	47.7	15.1	22.1	2	10.8	1.0	9.8	5.4	6.3	2	49.7	2.0	47.7	24.9	32.4	18%	82%					18%	82%
S0220CPT	Full Depth	23	119.3	1.0	28.5	5.2	6.4	11	32.0	1.0	7.9	2.9	2.4	11	81.7	1.0	28.5	7.4	8.6	28%	72%	28%	72%				
	Above 35ft	10	36.2	1.0	7.9	3.6	2.4	4	11.8	1.0	7.9	3.0	3.3	5	18.7	1.0	5.9	3.7	1.9	39%	61%			39%	61%		
	Below 35ft	13	83.2	1.0	28.5	6.4	8.2	7	20.2	1.0	6.9	2.9	2.1	6	63.0	1.0	28.5	10.5	10.9	24%	76%					24%	76%
S0221CPT	Full Depth	23	115.2	1.0	30.5	5.0	7.2	11	21.2	1.0	6.9	1.9	1.8	11	87.6	1.0	30.5	8.0	9.6	19%	81%	19%	81%				
	Above 35ft	8	44.9	1.0	14.8	5.6	4.7	3	5.9	1.0	3.0	2.0	1.0	4	32.5	2.0	14.8	8.1	5.5	15%	85%			15%	85%		
	Below 35ft	15	70.4	1.0	30.5	4.7	8.4	8	15.3	1.0	6.9	1.9	2.0	7	55.1	1.0	30.5	7.9	11.7	22%	78%					22%	78%
S0222CPT	Full Depth	20	93.4	0.5	26.6	4.7	6.0	9	20.7	1.0	5.9	2.3	2.1	10	66.4	0.5	26.6	6.6	7.8	24%	76%	24%	76%				
	Above 35ft	10	39.8	1.0	11.8	4.0	3.4	4	9.8	1.0	5.9	2.5	2.3	5	23.6	1.0	11.8	4.7	4.2	29%	71%			29%	71%		
	Below 35ft	10	53.6	0.5	26.6	5.4	7.9	5	10.8	1.0	5.9	2.2	2.1	5	42.8	0.5	26.6	8.6	10.5	20%	80%					20%	80%
S0225CPT	Full Depth	30	116.7	0.5	32.5	3.9	6.0	14	42.7	1.0	13.1	3.0	3.2	15	69.0	0.5	32.5	4.6	8.0	38%	62%	38%	62%				
	Above 35ft	10	38.5	1.0	13.1	3.8	3.7	4	22.0	1.0	13.1	5.5	5.5	5	11.5	1.0	4.9	2.3	1.6	66%	34%			66%	34%		
	Below 35ft	20	78.2	0.5	32.5	3.9	6.9	10	20.7	1.0	3.9	2.1	1.0	10	57.6	0.5	32.5	5.8	9.6	26%	74%					26%	74%
S0226CPT	Full Depth	23	122.5	1.0	36.4	5.3	7.7	11	28.1	1.0	9.8	2.6	2.7	11	88.6	1.0	36.4	8.1	10.2	24%	76%	24%	76%				
	Above 35ft	8	38.3	1.0	10.8	4.8	3.8	3	16.7	2.0	9.8	5.6	4.0	4	15.7	1.0	10.8	3.9	4.6	52%	48%			52%	48%		
	Below 35ft	15	84.2	1.0	36.4	5.6	9.2	8	11.3	1.0	3.0	1.4	0.8	7	72.8	2.0	36.4	10.4	12.1	13%	87%					13%	87%

Table A4.8-1

CPT-Based Assessment of Interbedding of Coarse and Fine Grained Layers in Native Soils of Tulare County

CPT ID	Depth Range	LAYERS: ALL SOILS (COARSE & FINE)						LAYERS: COARSE						LAYERS: FINES						PERCENTAGE		% - Full Depth		% - Above 35ft		% - Below 35ft	
		COUNT	SUM	MIN	MAX	MEAN	STDEV	COUNT	SUM	MIN	MAX	MEAN	STDEV	COUNT	SUM	MIN	MAX	MEAN	STDEV	COARSE	FINE	COARSE	FINE	COARSE	FINE	COARSE	FINE
S0230CPT	Full Depth	22	102.1	0.5	32.3	4.6	6.7	11	21.3	0.5	5.1	1.9	1.5	10	74.6	2.0	32.3	7.5	9.1	22%	78%	22%	78%				
	Above 35ft	10	34.9	1.0	6.2	3.5	1.5	5	14.9	1.0	5.1	3.0	1.6	4	13.8	3.0	3.9	3.4	0.6	52%	48%			52%	48%		
	Below 35ft	12	67.3	0.5	32.3	5.6	9.0	6	6.4	0.5	2.0	1.1	0.5	6	60.9	2.0	32.3	10.1	11.3	10%	90%					10%	90%
S0237CPT	Full Depth	13	99.7	1.0	23.6	7.7	7.4	6	14.8	1.0	5.9	2.5	2.0	6	79.2	3.0	23.6	13.2	7.5	16%	84%	16%	84%				
	Above 35ft	7	50.0	1.0	23.6	7.1	7.5	3	10.8	1.0	5.9	3.6	2.5	3	33.5	3.0	23.6	11.2	11.0	24%	76%			24%	76%		
	Below 35ft	6	49.7	1.0	18.2	8.3	7.8	3	3.9	1.0	2.0	1.3	0.6	3	45.8	12.8	18.2	15.3	2.7	8%	92%					8%	92%
S0239CPT	Full Depth	8	99.7	1.0	57.6	12.5	19.1	3	5.9	2.0	2.0	2.0	0.0	4	88.1	1.0	57.6	22.0	24.6	6%	94%	6%	94%				
	Above 35ft	6	40.1	1.0	16.7	6.7	6.6	2	3.9	2.0	2.0	2.0	0.0	3	30.5	1.0	16.7	10.2	8.2	11%	89%			11%	89%		
	Below 35ft	2	59.5	2.0	57.6	29.8	39.3	1	2.0	2.0	2.0	2.0	NA	1	57.6	57.6	57.6	57.6	NA	3%	97%					3%	97%
S0241CPT	Full Depth	13	99.9	1.0	32.9	7.7	8.2	6	17.7	1.0	4.9	3.0	1.5	6	77.2	5.9	32.9	12.9	10.0	19%	81%	19%	81%				
	Above 35ft	7	39.5	2.0	10.8	5.6	2.8	3	10.8	2.0	4.9	3.6	1.5	3	23.6	5.9	10.8	7.9	2.6	31%	69%			31%	69%		
	Below 35ft	6	60.5	1.0	32.9	10.1	11.9	3	6.9	1.0	3.9	2.3	1.5	3	53.6	9.8	32.9	17.9	13.0	11%	89%					11%	89%
AVGS*	**Full Depth**	**21.8**	**112.3**	**0.8**	**25.9**	**5.6**	**6.6**	**10.5**	**38.4**	**1.0**	**10.5**	**3.5**	**3.1**	**10.3**	**67.8**	**1.2**	**25.3**	**7.7**	**8.3**	**36%**	**64%**	**36%**	**64%**				
	Above 35ft	**8.5**	**40.0**	**1.1**	**15.0**	**5.2**	**5.0**	**3.7**	**13.6**	**1.7**	**7.4**	**4.0**	**2.9**	**3.8**	**20.4**	**2.0**	**12.9**	**6.5**	**6.0**	**42%**	**58%**			**42%**	**58%**		
	Below 35ft	**13.3**	**72.3**	**0.9**	**23.6**	**6.7**	**7.9**	**6.8**	**24.8**	**1.1**	**8.6**	**3.5**	**2.9**	**6.5**	**47.5**	**3.8**	**22.9**	**10.1**	**8.1**	**33%**	**67%**					**33%**	**67%**

* Averages of the mean layer thickness over all CPTs, for 'ALL SOILS', range between 4.7 and 5.4ft, with a mean of 5.1ft.

= Significant outlier for boundary selection criteria.

RED = MORE THAN ONE STANDARD DEVIATION BELOW MEAN

ORANGE = MORE THAN ONE STANDARD DEVIATION ABOVE MEAN

		PERCENTAGE COARSE	PERCENTAGE FINE	% - Full Depth COARSE	% - Full Depth FINE	% - Above 35ft COARSE	% - Above 35ft FINE	% - Below 35ft COARSE	% - Below 35ft FINE
STANDARD DEVIATION	Full Depth	16%	16%	16%	16%				
	Above 35Fft	21%	21%			21%	21%		
	Below 35ft	19%	19%					19%	19%
	MIN					11%	89%	3%	97%
	MAX					90%	10%	70%	30%

		Manual Corrections:	COARSE	FINE
S0193CPT:	Point with 'outlier boundary' includes extra 40ft of fines Above 35ft. Correction puts all values closer to the site-wide mean.	Above 35ft	60%	40%
		Below 35ft	22%	78%
S0214CPT:	Point with 'outlier boundary' includes extra 25ft of fines Above 35ft. Correction puts Above 35ft values closer to the site-wide mean, but exacerbates divergence from the mean for values Below 35ft.	Above 35ft	44%	56%
		Below 35ft	11%	89%

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A5.0 References

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